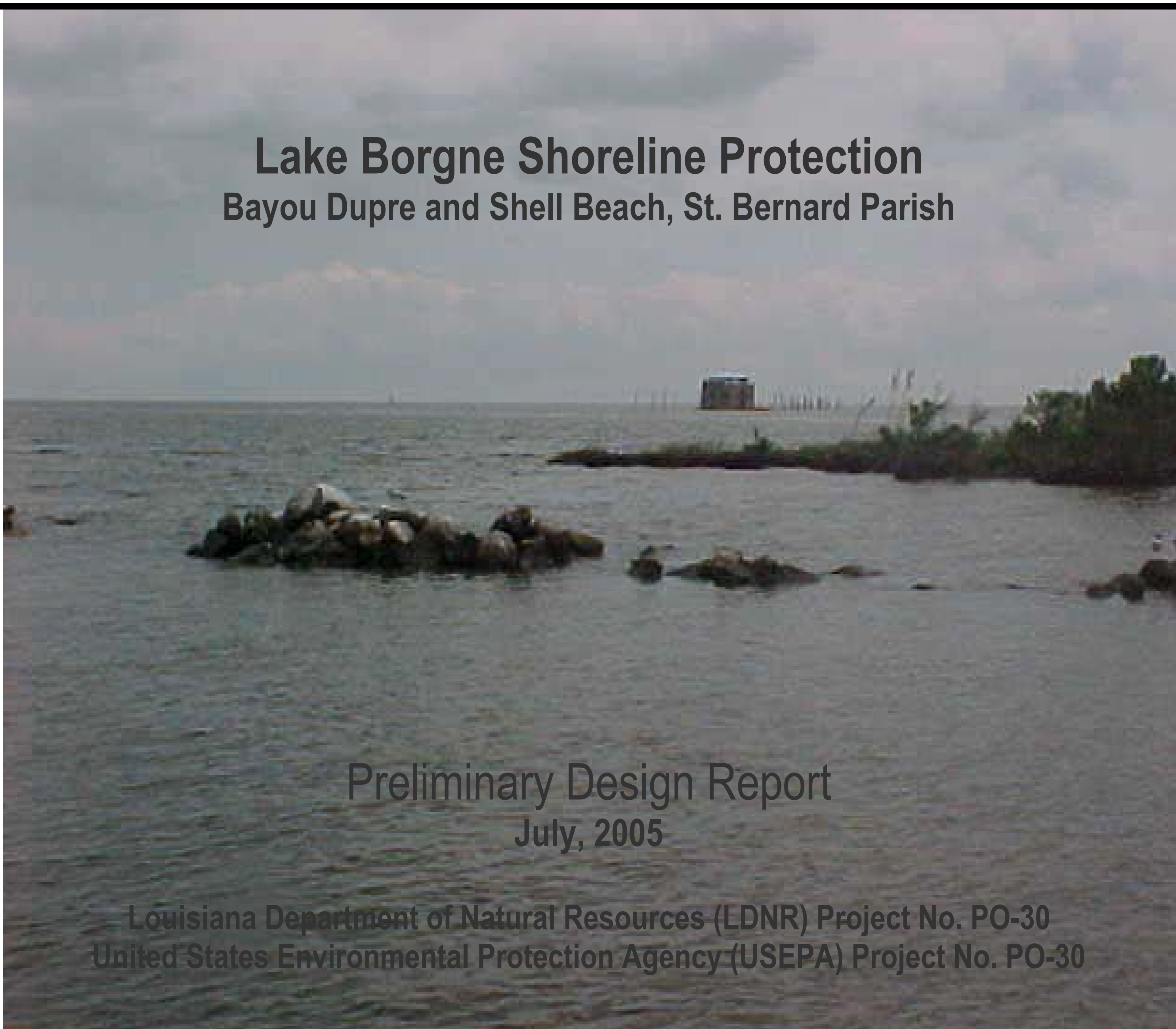

Lake Borgne Shoreline Protection

Bayou Dupre and Shell Beach, St. Bernard Parish

Preliminary Design Report

July, 2005

Louisiana Department of Natural Resources (LDNR) Project No. PO-30

United States Environmental Protection Agency (USEPA) Project No. PO-30

Coastal Wetlands Planning, Protection and Restoration Act (CWPPRA)

Table of Contents

1. INTRODUCTION

The Lake Borgne Shoreline Protection Project (herein referred to as PO-30) is located in the Pontchartrain Basin on the southern shoreline of Lake Borgne. The Louisiana Coastal Wetlands Conservation and Restoration Task Force (Task Force) designated PO-30 as part of the 10th Priority Project List. The United States Environmental Protection Agency Region 6 (EPA) was designated as the lead federal sponsor. The Louisiana Department of Natural Resources, Coastal Engineering Division (LDNR-CED) was selected by EPA to perform engineering and design for the project. Approval to proceed with engineering and design was granted at the January 2001 Task Force meeting. Funds for the project are provided through the Federal Coastal Wetlands Planning, Protection and Restoration Act (Public Law 101-646) and the local cost share is provided by the State of Louisiana's Wetlands Conservation Trust Fund.

The initial project provided lakeside protection only to the Old Shell Beach area. In April 2002, the Task Force combined the original project and funding with the Lake Borgne Shoreline Protection at Bayou Dupre (PO-31) from Priority Project List 11. The combined project (PO-30) is divided into two sections, Bayou Dupre and Shell Beach. The section at Shell Beach extends approximately 3.4 miles between Fort Bayou and Doulluts Canal, and the section at Bayou Dupre extends approximately 1.4 miles to the west and 1.2 miles to the east of Bayou Dupre (Figure 1).

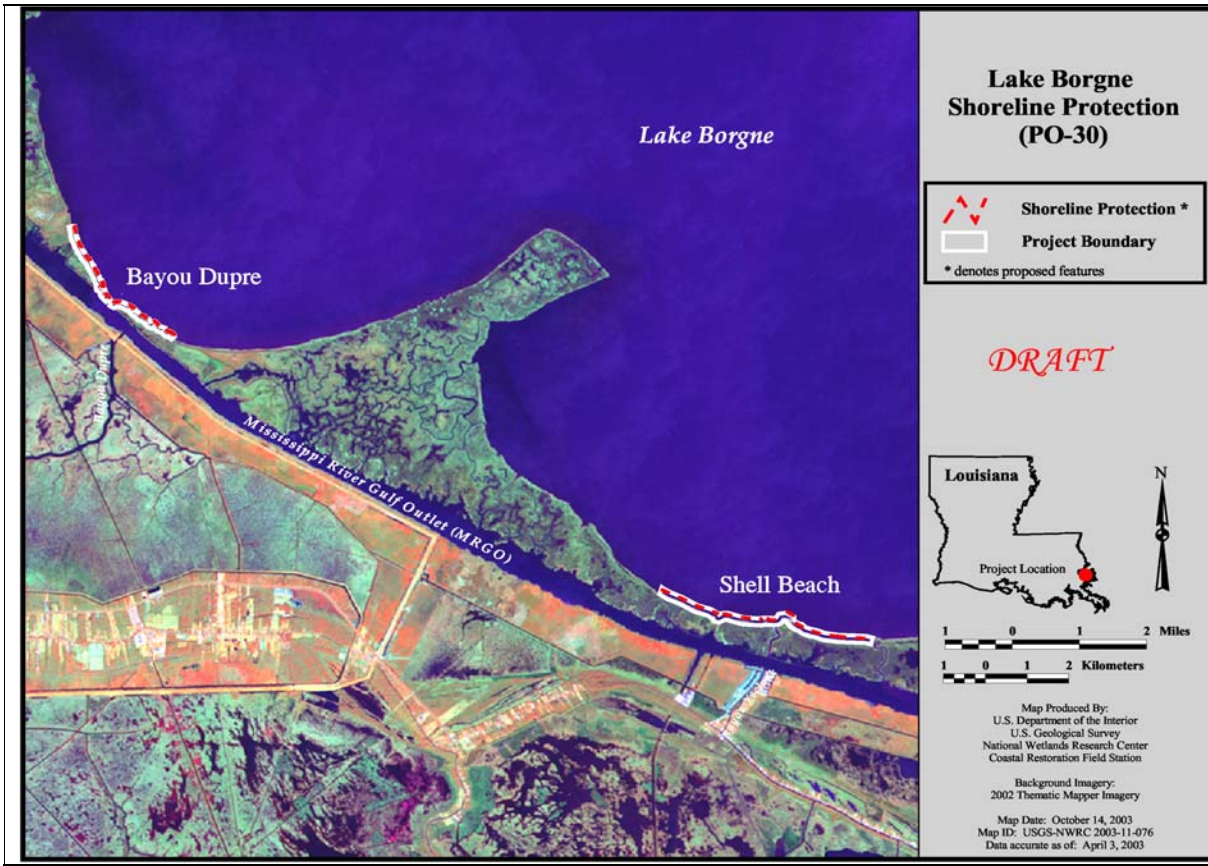

Figure 1 – PO30 Project Boundaries

The narrow strip of marsh which separates Lake Borgne from the Mississippi River Gulf Outlet (MRGO) is degrading at an estimated 9 feet per year at Shell Beach, and 10 feet per year at Bayou Dupre. This narrow strip of marsh also protects the coastal communities of Shell Beach, Yscloskey, and Hopedale from wave energy and tidal surge generated in Lake Borgne. The objectives of this project are to halt shoreline retreat and direct marsh loss along Lake Borgne, prevent further coalescence of the lake and MRGO, re-establish a sustainable lake rim, restore saline marsh habitat, and enhance fish and wildlife habitat.

The proposed solution is to construct a nearly continuous rock breakwater along the designated shoreline sections of Lake Borgne at Bayou Dupre and Shell Beach. At the mouth of Bayou Dupre, maintenance dredging within the MRGO has created an unnatural water depth. Therefore, a sheet pile structure or equivalent will tie the proposed shoreline breakwater into the existing offshore USACE rock breakwater along the MRGO. At Shell Beach, the proposed rock breakwater will tie into the existing rock breakwater which surrounds the perimeter of Fort Beauregard and the only openings in the breakwater will occur along the mouth of Bayou Yscloskey and across the Tennessee Gas Pipeline right-of-way. The design life for the project is 20 years.

A temporary flotation channel will also be excavated along the shoreline in order to facilitate construction and maintenance of the rock breakwater. The spoil will be deposited on the lakeside of the flotation channel and degraded back into the flotation channel after construction or maintenance of the rock breakwater is complete.

The project team, consisting of members of EPA, LDNR-CED, the St. Bernard Parish Council, and Coastal Zone Monitoring committee, performed an on-site kick-off meeting on March 8, 2001. Based on that meeting, a plan was developed to identify and address all of the project requirements. The engineering and design, environmental compliance, real estate negotiations, oyster lease acquisitions, and cultural resources investigations are currently at the 30% level of completion as required by the standard operating procedures.

2. SURVEYS

2.1. Topographic, Bathymetric and Magnetometer Surveys

In order to facilitate the design of the shoreline protection structure and associated flotation channel, bathymetric, topographic and magnetometer surveys were performed for Shell Beach on February 25, 2002 by BFM Corporation, L.L.C., and on March 21, 2005 by Sigma Consulting Group, Inc. (Appendix A), and for Bayou Dupre on January 13, 2004 and on March 21, 2005 by Sigma Consulting Group, Inc., (Appendix B). A magnetometer survey near the former naval base on Bayou Yscloskey at Lake Borgne was performed by Earth Search, Inc., on March 17, 2005.

The survey baseline for Shell Beach was established along the shoreline extending from the east bank of Fort Bayou to the west bank of Doullut's Canal. The survey transects intersect the baseline at 1000 foot intervals and extend perpendicular into Lake Borgne from 25 feet onshore to the approximate -7.0 foot contour, except at the middle an outermost transects where they extend to the -8.0 foot contour. Upland and shallow water areas were shot using conventional level soundings. Deepwater areas were shot using a fathometer and RTK positioning.

In order to identify potentially live ordnance along the immediate shoreline of the former naval facility located east of Bayou Yscloskey at Lake Borgne, a separate magnetometer survey was performed. One hundred and twenty-one anomalies were detected by the survey. Individual ordnance, if present, was masked by the magnetic inflections of existing large-scale structures. According to the Formerly Used Defense Sites 2002 Properties list by the United States Corps of Engineers, no hazardous potential was found at the officially closed site.

The survey baseline for Bayou Dupre was established along the shoreline extending approximately 1.6 miles to the west and 1.2 miles to the east of the bayou. The survey transects intersect the baseline at 500 foot intervals within the bayou and 1000 foot intervals thereafter, and extend perpendicular into Lake Borgne from 25 feet onshore to the approximate -8.0 foot contour in Lake Borgne. An additional transect was added along an approximate 200 foot section extending between the existing rock breakwaters along the MRGO located immediately west of the bayou. Upland and shallow water areas were shot using conventional level soundings. Deepwater areas were shot using a fathometer and RTK positioning.

2.2. Secondary Monuments

Prior to performing the topographic and bathymetric survey of the project areas, permanent secondary monuments were installed at Shell Beach and Bayou Dupre. "PO-30-SM-01" was installed on the south bank of the MRGO at Bayou Yscloskey having coordinates of 29°56'10.33674"N, 89°50'08.86486"W. "SHELL BEACH 2002" was installed at the northwest end of Louisiana State Highway 46 having coordinates of 29°51'17.18441"N, 89°40'41.00787"W. These monuments were established primarily for this project but are also now part of the LDNR secondary GPS network. The data sheets for these monuments are provided in Appendices C and D.

3. WIND ANALYSIS

NOAA Station 42007 was selected to gather historical wind data due to availability and close proximity to the project area. It is located in the Gulf of Mexico (30°05'24"N; 88°46'12"W), approximately 22 miles south-southeast of Biloxi, Mississippi, and approximately 40 miles northeast of the project area.

Based on statistical analysis of the hourly wind data available from 1993 to 2002, the 90th percentile wind direction was determined to be 39.69˚ north-northeast as shown in Figure 2. The 90th percentile wind speed associated with the 90th percentile wind direction was calculated to be 23.3 miles per hour.

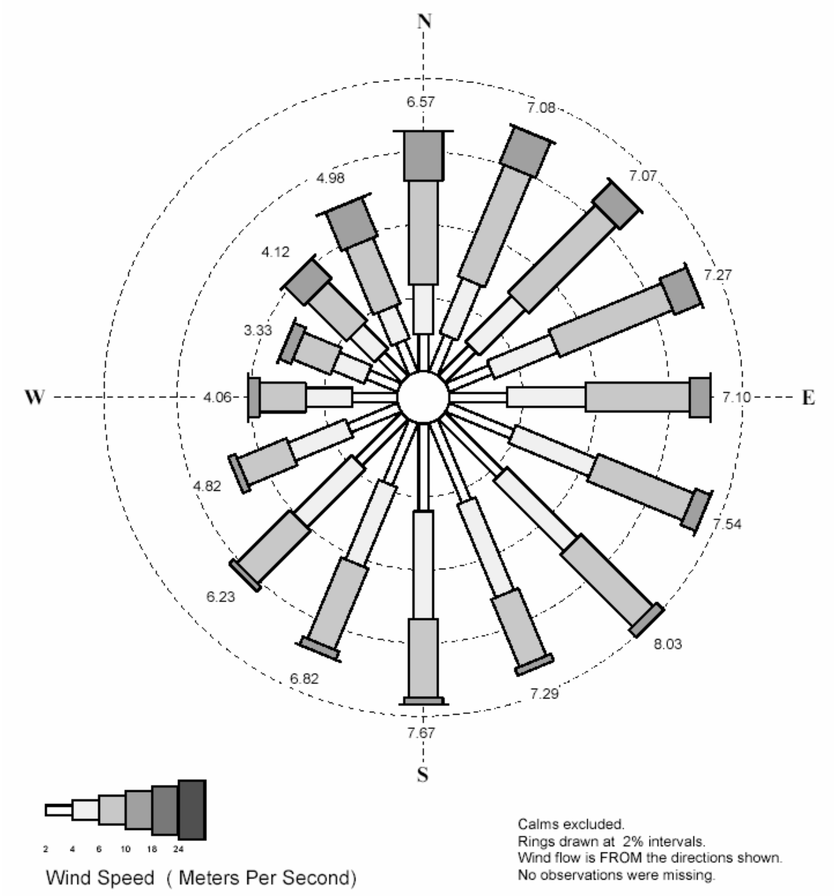

Figure 2 – Wind Rose for NOAA Station 42007, 1993-2002

4. HYDRAULICS

4.1. Historic Water Levels

USACE Gage Station 85800 was selected to gather historical water level records due to its close proximity to the project area and database availability. It is located on Bayou Yscloskey at 29°51'00"N; 89°41'00"W, approximately 200 feet southwest of the junction with the MRGO. Based upon historical water level records from 1993 to 2002 the mean high water (MHW), mean water level (MWL), and the mean low water level (MLW) were determined as shown in Table 1. The gage is referenced to NGVD29 but all values were corrected by -0.72 feet to the NAVD88 datum by the USACE.

DATUM	NORTHING	EASTING	NGVD 29	NAVD 88	CHANGE
	(U.S. FEET)	(U.S. FEET)	(U.S. FEET)	(U.S. FEET)	(U.S. FEET)
MHW	496,520.60	3,805,331.73	1.90	1.18	-0.72
MW	496,520.60	3,805,331.73	1.24	0.52	-0.72
MLW	496,520.60	3,805,331.73	0.57	-0.15	-0.72

Table 1 – Water Level Elevations at USACE Gage Station 85800, 1993-2002

4.2. Setup

The setup for Lake Borgne at Bayou Dupre and Shell Beach was determined using the 90th percentile water and wave conditions from the historical records. The average recorded water level associated with the 90th percentile wind speed and direction is 1.67 feet (0.5m) NAVD88. This value minus the mean high water level yields a setup of 0.49 feet (0.15 m).

4.3. Deep Water Wave Hind Casting

According to NOAA Nautical Chart #11371 (1989), the average depth of Lake Borgne is approximately 7 feet in the western lobe and 9 feet in the eastern lobe. For Shell Beach, the longest fetch associated with the 90th percentile wind direction and continuous 9 foot water depth is 22 miles as shown in Figure 3. For Bayou Dupre the longest fetch associated with the 90th percentile wind direction and continuous 7 foot water depth is 7.5 miles as shown in Figure 4.

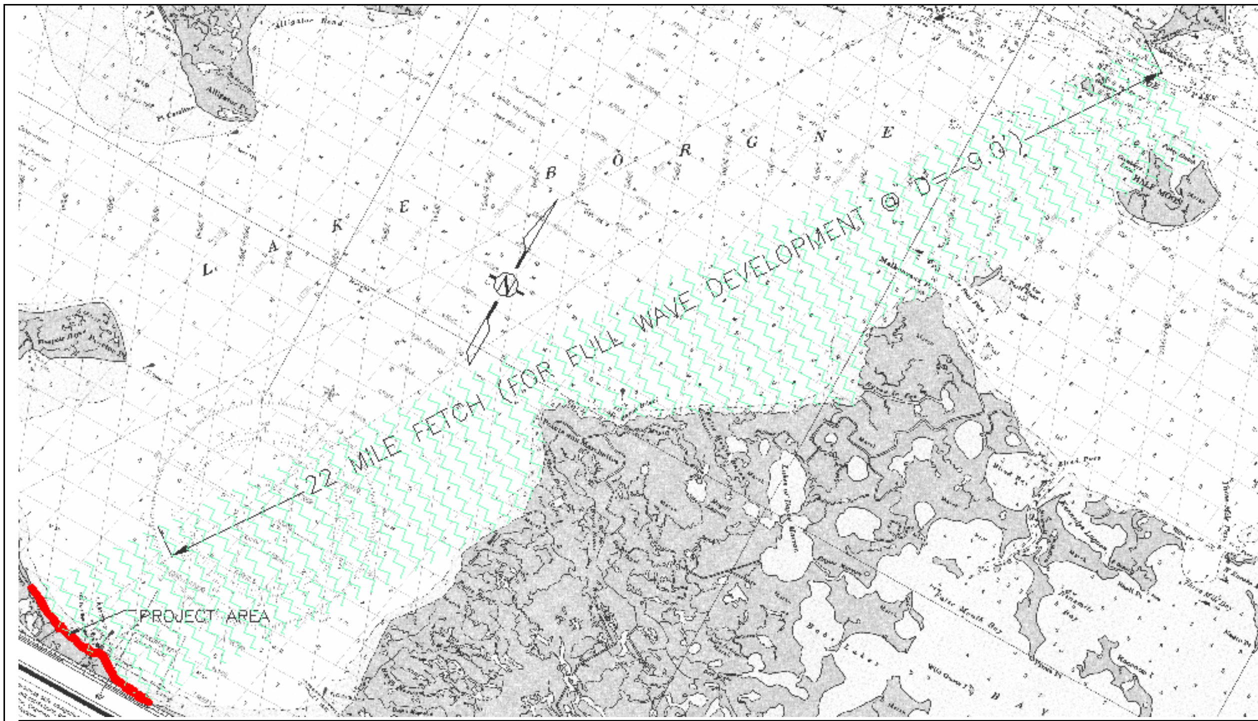

Figure 3 (NTS) – Maximum Reach for Wind Generated Wave at Shell Beach

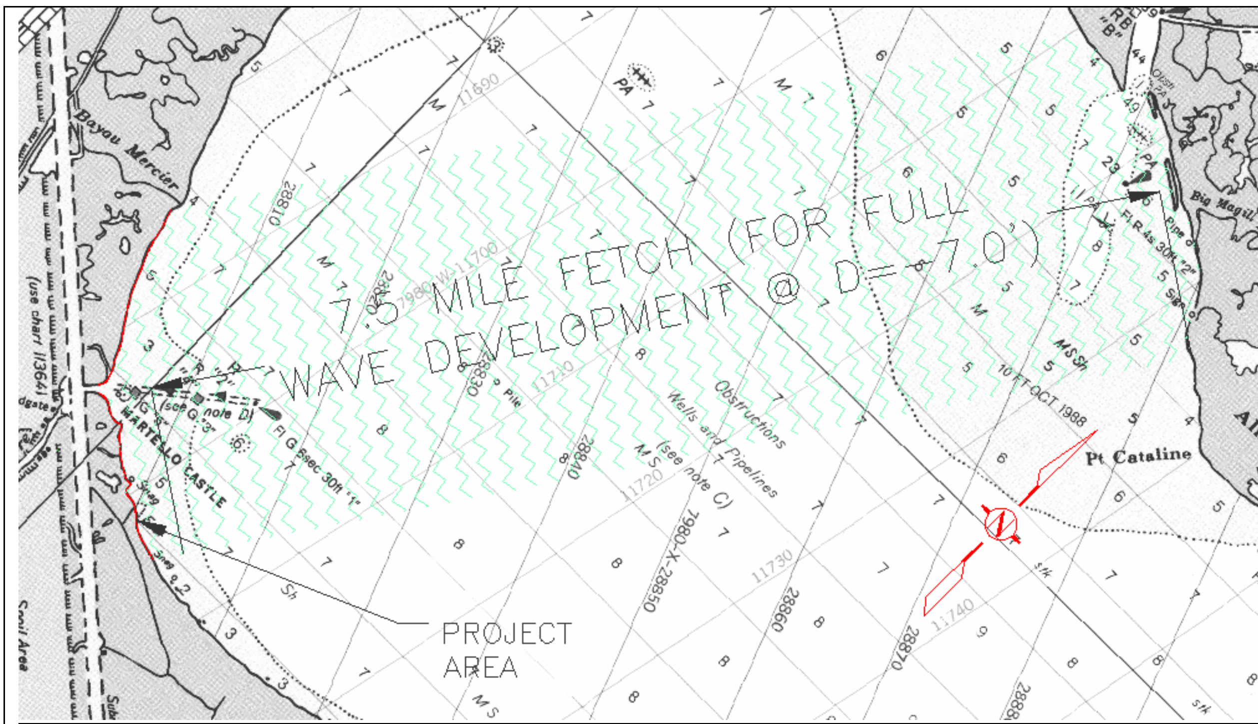

Figure 4 (NTS) – Maximum Reach for Wind Generated Wave at Bayou Dupre

Using the deep water nomograms in Figure II-2-23 of the U.S. Army Corps of Engineers Coastal Engineering Manual (USACE CEM), the deep water wave height and period for Shell Beach were determined to be 0.9 meters (2.9 feet) and 3.5 seconds, respectively (Figure 5). For Bayou Dupre, the relative deep water wave height and period were determined to be 0.5 meters (1.6 feet) and 2.4 seconds, respectively (Figure 6). The values for deep water wave height from the nomograms are relative to still water elevation and represent the wave profile from crest to trough. The deepwater waves generated for both areas were not fetch or shallow water limited.

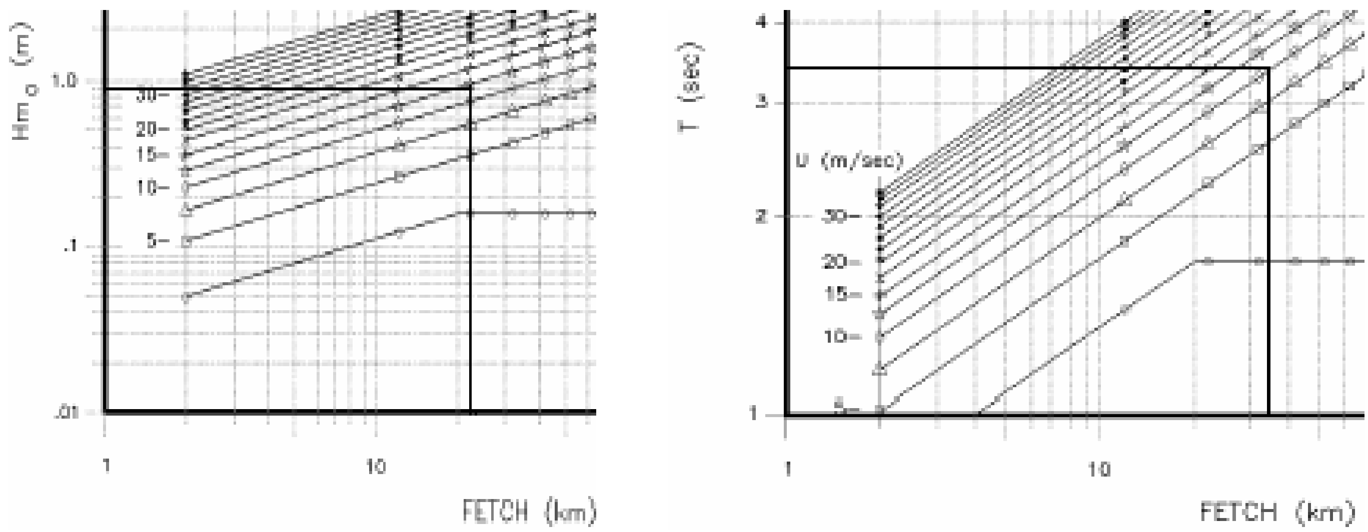

Figure 5 – Deep Water Wave Nomographs for Lake Borgne at Shell Beach

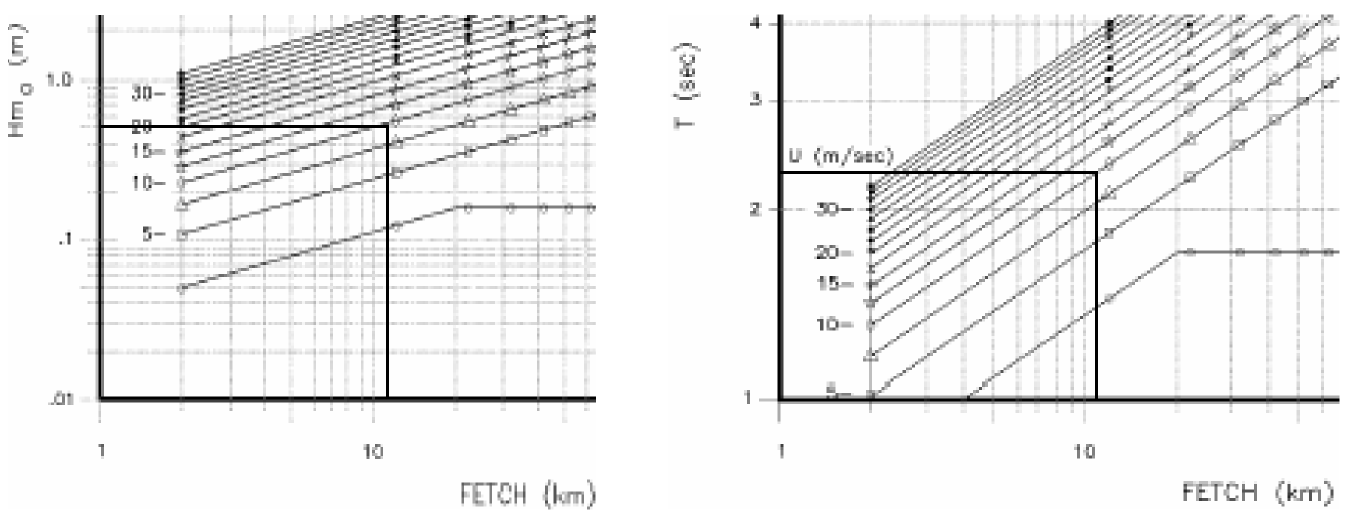

Figure 6 – Deep Water Wave Nomographs for Lake Borgne at Bayou Dupre

For this design, the components of the absolute deep water wave height include the setup, mean high water level, and relative deep water wave height shown in the nomograms. Therefore, for Bayou Dupre, the absolute deep water wave height is 0.49 ft + 1.18 ft + 0.8 ft = 2.47 ft NAVD88.

For Shell Beach, the absolute deep water wave height is 0.49 ft + 1.18 ft + 1.34 ft = 3.01 ft NAVD88.

4.4. Wave Transformation

As a deep water wave propagates shoreward along increasing bathymetry, it loses energy, and therefore height due to frictional forces. These frictional forces are caused by the reflection and refraction of the wave with the bottom surface. Calculations were performed based on the methodologies in Chapter II of the USACE CEM to determine the height of the 90th percentile wind generated wave in deep water as it is transformed onshore at Bayou Dupre and Shell Beach (Table 2). For Bayou Dupre, it was determined that the 90th percentile wind generated wave would break between the 0.0 and 1.0 foot NAVD88 contours assuming an initial wave reflectivity angle of 25 degrees. For Shell Beach, it was determined that the 90th percentile wind generated wave would break between the -1.0 and 0.0 foot NAVD88 contours assuming an initial wave reflectivity angle of 11 degrees.

Contour	**Wave Height @ Bayou Dupre**			**Wave Height @ Shell Beach**		
(ft NAVD88)	H/2 (ft)	Water Type	h_{mhw}+Setup+H/2 (ft NAVD88)	H/2 (ft)	Water Type	h_{mhw}+Setup+H/2 (ft NAVD88)
-7	0.77	Transition	2.45	1.35	Transition	3.01
-6	0.76	Transition	2.43	1.36	Transition	3.03
-5	0.75	Transition	2.42	1.37	Transition	3.05
-4	0.74	Transition	2.42	1.40	Transition	3.07
-3	0.74	Transition	2.41	1.43	Transition	3.10
-2	0.74	Transition	2.42	1.43	Transition	3.10
-1	0.76	Transition	2.43	1.04	Transition	2.72
0	0.50	Transition	2.17	0.50	Shallow	2.17
1	0.20	Shallow	1.87	0.20	Shallow	1.87

Table 2 – Deep Water Wave Transformation

4.5. Wave Run-up

The maximum height to which a breaking wave will run up onto the rock breakwater cannot be calculated using current methodologies. Instead, in order remain conservative, the minimum breakwater height required to provide protection against the 90th percentile wind generated and breaking wave is taken as the sum of the setup, mean high water level and the wave height corresponding to the design contour. For example, at Bayou Dupre and Shell Beach, approaching waves will break prior to reaching the rock breakwater if it is placed at edge of the shoreline (Approximate +0.5 ft NAVD88 contour) at mean water level (+0.52 ft NAVD88). For this case the highest 90th percentile breaking wave height along both of the reaches is calculated to be approximately 2.0 ft NAVD88. The crown height of the chosen shoreline protection feature must maintain this elevation in order to provide optimum performance throughout the 20 year design life of the project.

5. GEOTECHNICAL INVESTIGATION

5.1. Soils Investigation

A total of twenty-four subsurface borings were drilled along the shoreline of the project area beginning on February 17, 2002 by Louis J. Capozzoli & Associates, Inc (LJCA). Fourteen borings were drilled near Shell Beach (Figure 7) and ten borings were drilled near Bayou Dupre (Figure 8). The borings ranged in depth from 15 to 50 feet, and were sampled continuously to the 10 foot depth, and on 5 foot centers thereafter.

Figure 7 – Geotechnical Borings Near Shell Beach

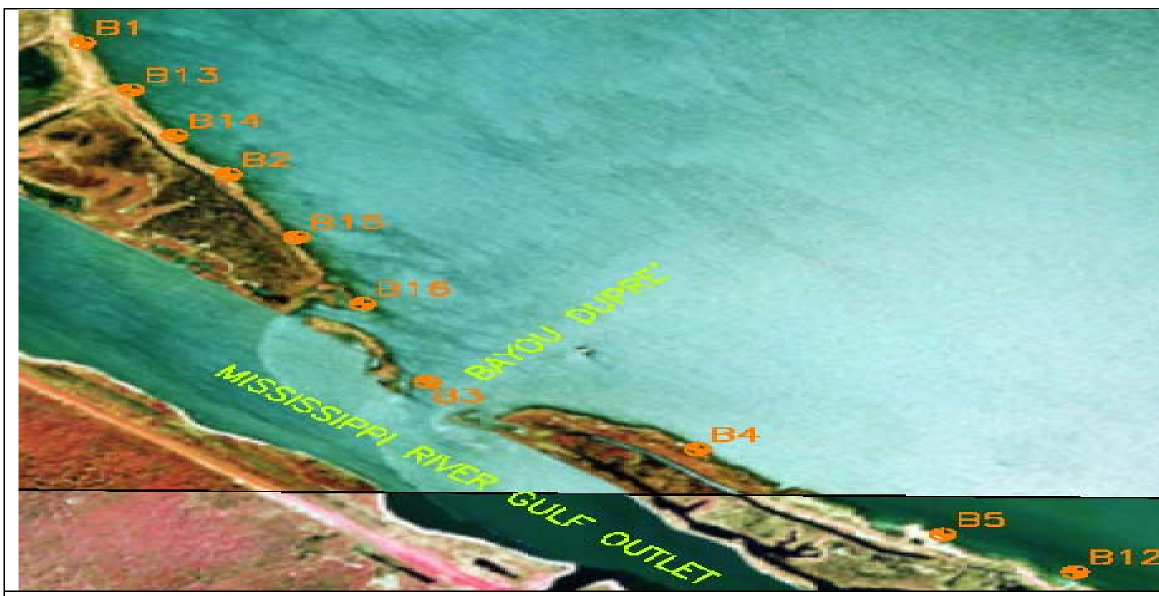

Figure 8 – Geotechnical Borings Near Bayou Dupre

The soils along the southern shoreline of Lake Borgne are generally very soft organic clays, peats and clays near the surface followed by several feet of very soft clays and silts. The shear strength and bearing capacity generally increases from the west to east along the project boundary.

Selected soil samples were tested in the laboratory for classification, strength, and compressibility. Analyses for settlement, bearing capacity and slope stability were performed for eight different rock breakwater sections (Table 3). The sections varied by type of material (250 lb. rock or lightweight aggregate), cross section, and depth of placement. The design elevation for the crown of all of the sections was set at +2.0 ft NAVD88 based on preliminary hydraulics information. The alignment for seven of the sections was based on offshore conditions in 2 feet of water. Only Section #8 was aligned with the lake ward toe located onshore at mean water elevation. All of the sections included nonwoven geotextile fabric and geogrid composite as support for the base. A detailed summary of the investigation is presented in the geotechnical report.

Section #	Contour (Ft NAVD88)	Crown Height (Ft NAVD88)	Crown Width (Ft)	Side Slopes H:V	Vertical Composition
1	-2	+2	4	2:1	4 ft stone
2	-2	+2	4	2:1	4 ft aggregate and stone
3	-5	+2	4	2:1	7 ft stone
4	-6	+2	4	2:1	8 ft stone
5	-2	+2	Multiple Furrow	2:1	4 ft aggregate and stone
6	-15	+2	4	2:1	17 ft aggregate and stone
7	-6	+2	Multiple Furrow	2:1	8 ft aggregate and stone
8	0	+2	4	2:1	4 ft stone

Table 3 – Design Sections from Geotechnical Report

5.2. Subsidence and Sea Level Rise

The combined subsidence and eustatic sea level rise rate for Lake Borgne is predicted to be 18 in/century, or a total of 3.6 inches over the 20 year design life of the project (EPA 1995). This rate was used to calculate the overall long term settlement rates of the rock breakwater sections.

5.3. Consolidation and Immediate Settlement

The LGCA geotechnical report evaluated the immediate (undrained) and consolidation (long-term) settlement rates for the eight alternative rock breakwater sections in order to determine the optimum breakwater section for the given soil conditions. The consolidation settlement rates varied between 0.5 to 53 inches within the 20 year design life of the project, but all of the alternatives were expected to reach a 95% degree of consolidation within this time period. The immediate settlement was estimated to be approximately 20% of the consolidation settlement.

The section in alternative #8 produced the smallest settlement rate among all of the eight alternatives considered. This section was aligned onshore at the 0 ft NAVD88 contour and consisted of class 250 lb rock, a 2 foot crown height, and 2:1 side slopes. The final settlement for this alternative varied based on subsurface conditions between 7 to 23 inches over the 20 year design life of the project.

Additional alternatives were evaluated at the +0.5 ft NAVD88 contour by LDNR/CED in order to optimize the design of the rock breakwaters. In order to evaluate the variability in settlement across the project area, the borings were separated into two sections, “Weak” and “Strong” soils according to shear strength profiles. Borings 8 and 9 represent the median of the “Strong” sections while borings B2 and B7 were selected to represent the “Weak” sections. The locations of these sections relative to the project area are shown in Figures 9 and 10.

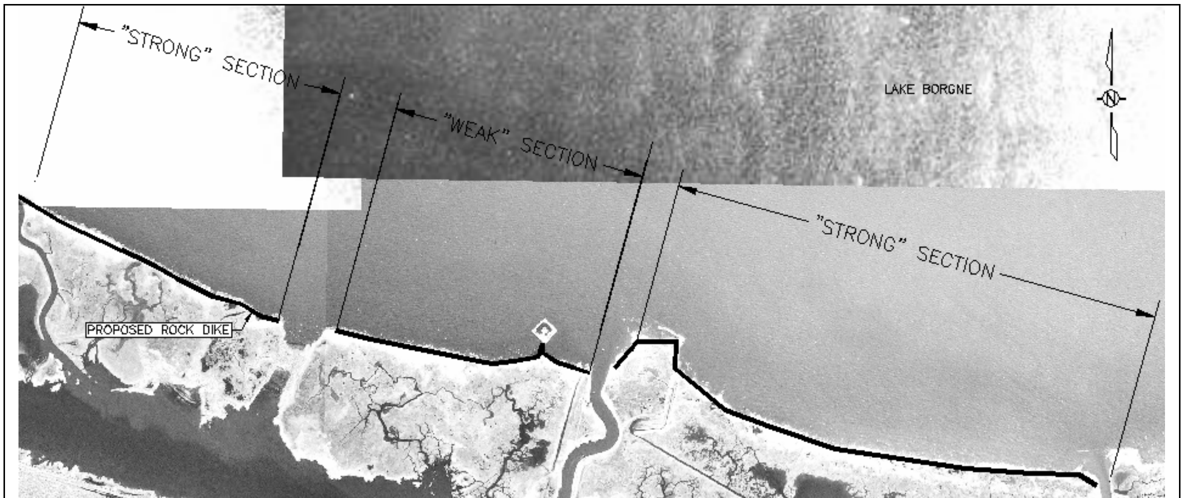

Figure 9 (NTS) – “Weak” and “Strong” Soil Settlement Sections at Shell Beach Section

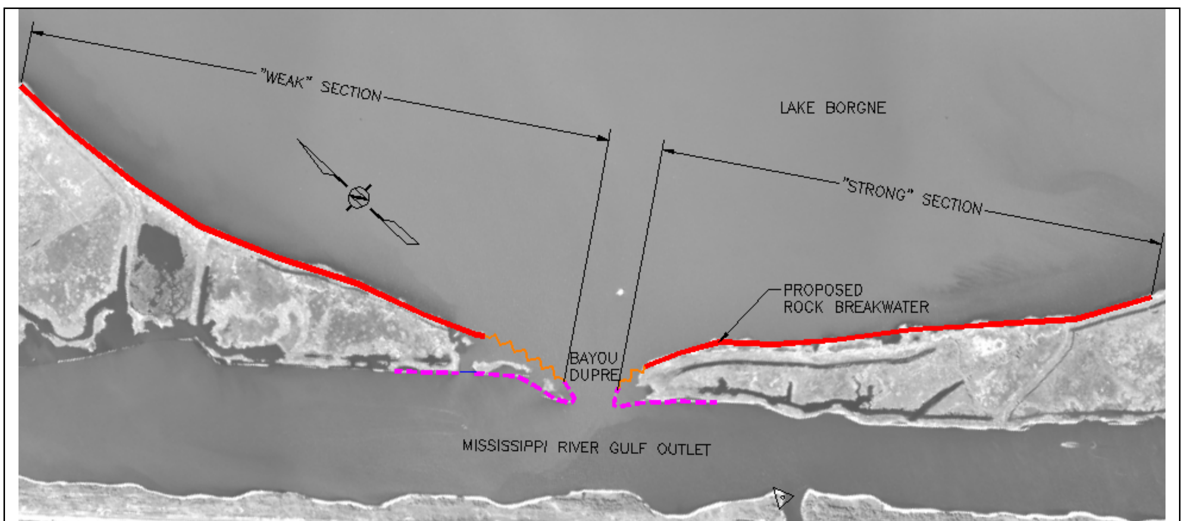

Figure 10 (NTS) – “Weak” and “Strong” Soil Settlement Sections at Bayou Dupre Section

Analysis of the “Weak” soil profile assumed the recent soils above the Pleistocene soils are normally consolidated. The “Strong” soil profile assumed the recent soils have experienced a minor amount of overconsolidation and generally contain better engineering properties.

The time rates of consolidation for both the “Weak” and “Strong” profiles were estimated using coefficients of consolidation (c_v). The “low” c_v values were determined from laboratory testing. The “High” c_v values are 10 times greater than the “Low” c_v values in order to assess the possibility that the field c_v values are greater than the laboratory (“Low”) values. Laboratory tests often do not reflect existing macro-level features that facilitate the dissipation of excess pore water pressures in the field.

Three lift cycles will be required to maintain the crown height of the rock breakwater at the optimum design height of +2.0 ft NAVD88 for the “Weak” sections over the 20 year design life of the project. The results of the “High” coefficient of consolidation were selected in order to be more conservative in the design approach. A combination of geogrid and geotextile will be placed beneath the footprint (plus 3 feet on either side) of the breakwater in order to improve constructability, maintain the load more uniformly, and increase the factor of safety for shear strength to 1.38. The breakwater will be constructed to an initial crown elevation of +3.0 ft NAVD88 and experience an estimated 1.5 feet of immediate settlement. At day 30, the breakwater will be re-constructed to elevation +3.25 ft NAVD88. At year 1, a final maintenance lift will be placed to elevation +4.0 ft NAVD88. The estimated construction and maintenance lift cycles are shown graphically in Figure 11.

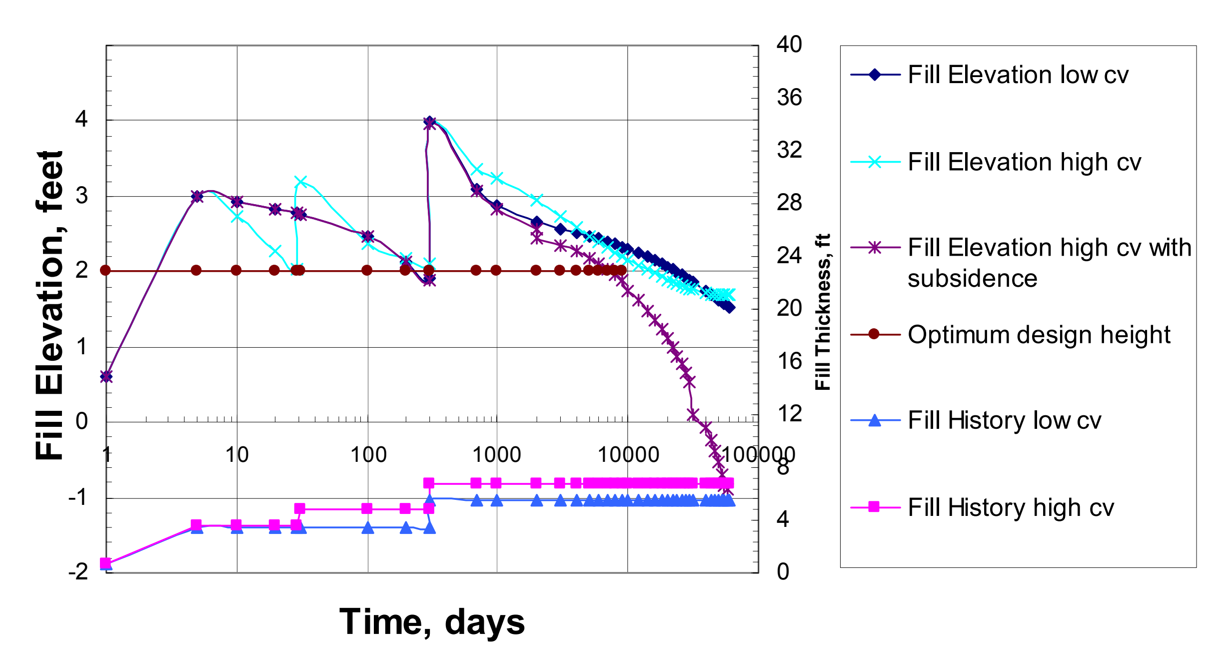

Figure 11 – Predicted Settlement for the “Weak” Breakwater Sections

For the “Strong” sections, one lift may be adequate to maintain the crown height of the rock breakwater at the optimum design height of +2.0 ft NAVD88 over the 20 year design life. Both the “Low” and “High” c_v cases are estimated to remain above this elevation over the 20 year design life of the project. A combination of geogrid and geotextile will be placed beneath the footprint (plus 3 feet on either side) of the breakwater in order to improve constructability, maintain the load more uniformly, and increase the factor of safety to 1.4 with respect to slope stability. The breakwater will be constructed to an initial crown elevation of +4.0 ft NAVD88 and may experience an estimated 2 inches of immediate settlement (Figure 12).

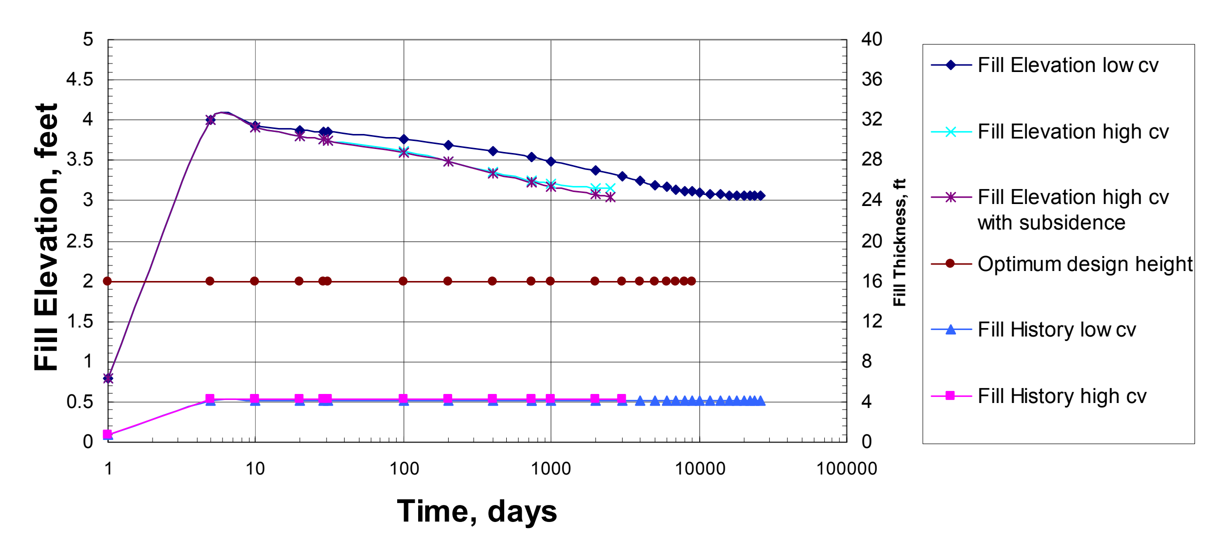

Figure 12 – Predicted Settlement for the "Strong" Breakwater Sections

5.4. Slope Stability and Bearing Capacity

The slope stability and ultimate bearing capacity of several alternative rock breakwater sections were originally analyzed in the geotechnical report with the alignment along at the 0 ft NAVD88 contour. Minimum factors of safety of 1.3 and 1.2 were used for calculating the slope stability and ultimate bearing capacity, respectively. The results of the analysis show a large variability across the entire project reach. Only the rock breakwater in alternative #8 (Crown Elevation +2.0 ft NAVD88) maintained the acceptable factors of safety across the entire project reach at the 0 ft NAVD88 contour.

Further analysis of additional alternatives was performed at the +0.5 ft NAVD88 contour subsequent to the geotechnical report. Assuming a stone density of 155 lb/ft^3 and porosity of 19%, the in-place unit weight of stone was estimated as follows:

$$\gamma_{STONE} = 155 \text{ lb/ft}^3 \times (1 - 0.19) = 125 \text{ lb/ft}^3$$

The maximum net allowable bearing pressure was estimated to be approximately 400 psf. The addition of geogrid/geotextile composite beneath the stone will load the soil more uniformly and increase the factor of safety relative to bearing capacity. With a goegrid/geotextile composite, the crown elevation of the "Weak" and "Strong" profiles can be set as high as +3.5 ft NAVD88 and +4.0 ft NAVD88, respectively.

The factor of safety with respect to slope stability was estimated for both the "Weak" and "Strong" profiles. The base elevation of the rock breakwater was set at +0.5 ft NAVD88 with H2:1V side slopes. The maximum crown elevations that can be achieved for the "Weak" and "Strong" profiles using geogrid are +3.0 ft NAVD88 and +4.0 ft NAVD88, respectively. The factors of safety for both profiles are greater than 1.35. Critical circular failures occur approximately 20 to 25 feet from the base of the "Weak" and "Strong" rock breakwater sections

(Figures 13 and 14). Taking into account the maximum available reach for a barge mounted track hoe, the distance from the lake ward toe of the rock breakwater to the flotation channel is therefore set at 50 feet in order to remain conservative.

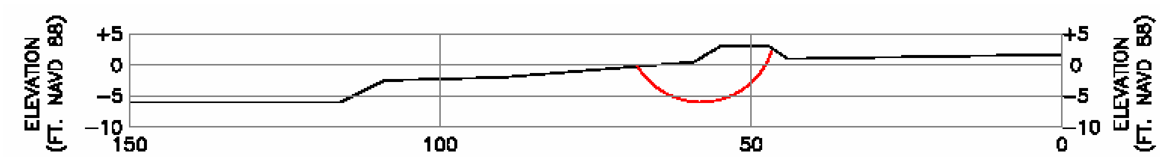

Figure 13 – Slope Stability Analysis of "Weak" Rock Breakwater (Crown +3.0' NAVD 88, Base +0.5' NAVD88)

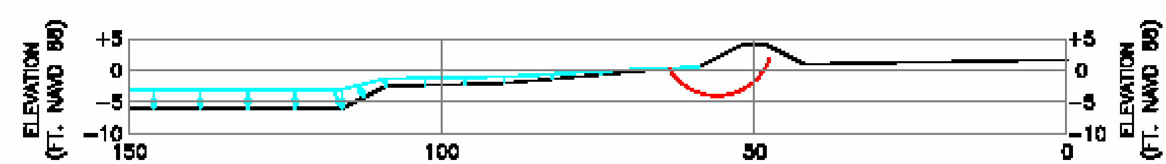

Figure 14 – Slope Stability Analysis of "Strong" Rock Breakwater (Crown +4.0' NAVD 88, Base +0.5' NAVD88)

6. DESIGN ALTERNATIVES

Four design alternatives were evaluated for use as protection along the shoreline of Lake Borgne at Shell Beach and Bayou Dupre; rock breakwaters, segmented concrete panels, steel sheet piles, and a combination of rock breakwaters and a back-to-back fiberglass sheet pile structure. A preliminary design was formulated for each of the design alternatives based on the minimum requirements of the project including the design wave height, existing bathymetry and topography, and consolidation settlement. A construction cost estimate was then calculated for each of the alternatives as shown in Attachment E.

Similar criteria were utilized in the preliminary design of the alternatives in order to maintain a consistent comparison of the cost estimates. All of the design alternatives used the same alignment along the approximate +0.5 ft NAVD88 contour except at the mouth of Bayou Dupre where it traverses along the shallowest route and connects to the existing USACE breakwaters on either side. The top elevations of the design alternative features were all set at the optimum design height of +2.0 ft NAVD88 at a minimum. At the mouth of Bayou Dupre, the top elevation was set at the deep water wave height of 2.5 ft NAVD88 due to the fact that the bathymetry actually deepens as it approaches the MRGO. For those design alternatives which included rock breakwaters, the crown elevations for the initial and maintenance lifts were adjusted for the bearing load of the rock profile, allowable bearing capacity of the existing soil, and preliminary settlement predictions.

For the segmented concrete panel alternative, 16 ft by 16 ft piles and 21 ft long panels with varying lengths based on the existing topography and bathymetry were utilized in the design. The total construction cost for segmented concrete panels is estimated to be approximately $14 million with a 15% contingency. This estimate excludes scour protection, flotation, and maintenance costs.

For the steel sheet pile alternative, a standard PZ-27 pile with varying lengths based on the existing topography and bathymetry were utilized in the design. The total construction cost for steel sheeting is estimated to be approximately $26.5 million with a 15% contingency. This estimate includes 35 foot soldier piles but excludes bracing, scour protection, flotation, and maintenance costs.

For the rock breakwater alternative, two lifts (three at the mouth of Bayou Dupre) were set at a crown elevation of +4.0 ft NAVD88 and crown width of 4 feet with 2 to 1 side slopes in order to maintain adequate protection against the deep water wave and consolidation settlement. The volume of rock required to construct the two lifts was nearly 300,000 tons. The total construction cost for the rock breakwater is estimated to be approximately $14.2 million with a 15% contingency. This estimate includes flotation and geogrid but excludes maintenance lifts due to variable consolidation settlement.

For the combination rock breakwaters and back-to-back fiberglass sheet pile structure alternative, the crown elevation of the breakwater was set at the optimum design elevation of +2.0 ft NAVD88. The structure consisted of a back-to-back fiberglass sheet pile structure set at a crown elevation of 2.5 ft NAVD88, interconnected by tie rods, backfilled with sand to mean water level, and capped with geofabric and 250 lb class stone. Composite fiberglass is comparable in strength when compared to steel, stronger and more durable than vinyl, and more economical than steel, rock and concrete. The total construction cost for the rock breakwaters and fiberglass sheeting is estimated to be approximately $11 million. This estimate includes scour protection, flotation, geogrid, settlement plates, warning signs, walers, tie rods, and backfill. Due to the expected longevity and lower construction costs for this alternative, the combination rock breakwaters and back-to-back fiberglass sheet pile structure was judged to be the preferred option as shown in Attachment E.

7. BREAKWATER DESIGN

As discussed in Section 6, the most cost effective shoreline protection feature is a semi-continuous rock breakwater along the +0.5 ft NAVD88 contour. Gaps will be provided at the mouth of Bayou Dupre, Bayou Yscloskey, and the pipeline crossing located west of Fort Beauregard. The breakwater will be designed to maintain its integrity against the design wave based on the 20 year design life of the project. Flotation and access channels will be provided in order to facilitate construction of the breakwater. The estimated materials quantities are provided in Attachment E. The final analysis and design of the breakwater will now be discussed.

7.1. Riprap Gradation

The size of the minimum stone class required by the breakwater to protect against the design wave was determined using the Hudson's Equation in Chapter VI of the USACE CEM as shown below:

$\mathbf{W_{50}}$ = Weight of Medium Stone (lb)

$$= \frac{(H^3)\hat{Y}s}{K_D(\hat{Y}s/\hat{Y}w-1)^3 \cot\alpha} \quad \textit{(Eq. VI-5-67)}$$

Where:
- H = 2.5 (Design wave height)
- K_D = 3.5 (Stability Coefficient, *Table VI-5-22*)
- $\hat{Y}s$ = 155 PCF (Weight of Stone)
- $\hat{Y}w$ = 62.4 PCF (Density of Water)
- α = 0.4 (2:1 Slope)

Using the deep water wave height of 2.5 ft as a conservative estimate at Bayou Dupre yields W_{50}=67 lbs. Using the deep water wave height of 3.2 ft as a conservative estimate at Shell Beach yields a W_{50}=140 lbs. Due to economy of scale, a class 250 lb stone was chosen for design and construction.

7.2. Minimum Crest Width

In order for the 250 lb class rock breakwater to withstand the force of the design wave, the minimum crest width was calculated from the guidelines in Chapter VI of the USACE CEM as shown below:

B = Minimum crest width (ft)

$$= n^*k_\Delta^*(W/w_a)^{1/3} \quad \textit{(Eq. VI-5-116)}$$

Where:
- n = 3.0 (Number of stones, typical)
- k_Δ = 1.0 (Layer coefficient, *Table VI-5-51*)
- W = 250 lb (Unit Weight of Primary Armor Unit)
- w_a = 155 PCF (Specific Weight of Rock)

The minimum crest width is calculated to be 3.5 ft. Adding a factor of safety of 0.5 foot to the design yields a crest width of 4 ft.

7.3. Minimum Layer Thickness

In order for the rock breakwater to withstand the force of the design wave, the minimum layer thickness was determined from the guidelines in Chapter VI of the USACE CEM as shown below:

r	= Minimum layer thickness (ft)		Where:			
1)	≥ 0.3m (0.98 Ft.)	= 0.9 ft	W_{50} =	Weight of 50% grade size	= 250 lb	
2)	$= 2*(W_{50}/w_a)^{1/3}$ *(Eq. VI-5-119)*	= 2.4 ft	w_a =	Specific weight of rock	= 155 PCF	
3)	$= 1.25*(W_{max}/w_a)^{1/3}$ *(Eq. VI-5-120)*	= 2.5 ft	W_{max} =	Max weight in gradation	= 250 lb	
r	= greatest of 1, 2 and 3	= 2.5 ft				

The minimum layer thickness of the rock is calculated to be 2.5 ft. Based upon the proposed geometry of a 4 ft. crest width, 3 or 4 ft NAVD88 crest height, +0.5 ft toe elevation, and 2:1 side slopes, this requirement is satisfied.

7.4. Typical Cross Section

The parameters used to set the typical cross sections for construction and maintenance lifts of the rock breakwaters include the crest height, crest width, side slope, and minimum layer thickness. As discussed in the previous sections, the toe of the breakwater is set at +0.5 ft NAVD 88. The side slopes are set at 2H:1V in conjunction with geogrid and geotextile underneath the foot print (+3 feet on either side) in order to maintain an adequate factor of safety for slope stability.

The crest height for the "Strong" condition is set at +4 ft NAVD88 for all of Reaches 2 and 4, and between Stations 10+00 to 55+52 of Reach 3. The typical cross section for the construction lift of the "Strong" rock breakwater is shown in Figure 15.

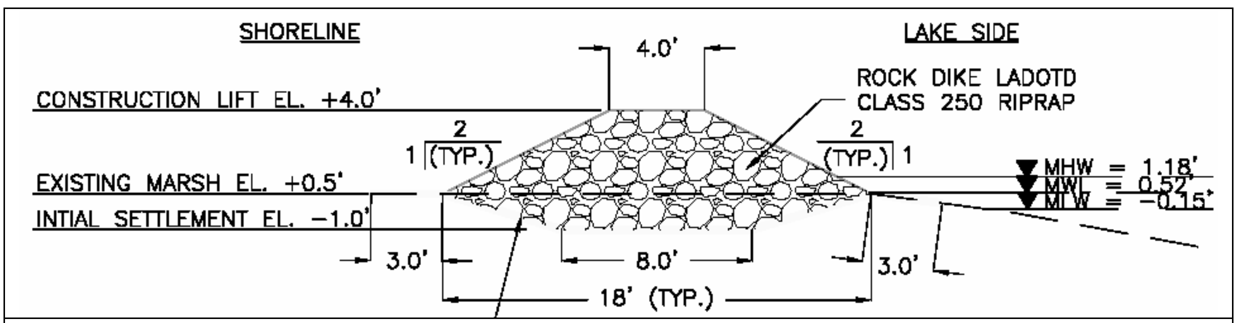

Figure 15 – Typical "Strong" Breakwater Section

The crest height for the "Weak" condition is set at +3 ft NAVD88 for the construction lift, +3.25 ft NAVD88 for the second (30 day) construction lift, and +4.0 ft NAVD88 for the maintenance lift (Year 1) along Reach #1 and between Stations 63+33 to 105+79 of Reach 3. The typical cross section for the construction and maintenance lifts of the "Weak" rock breakwater is shown in Figure 16.

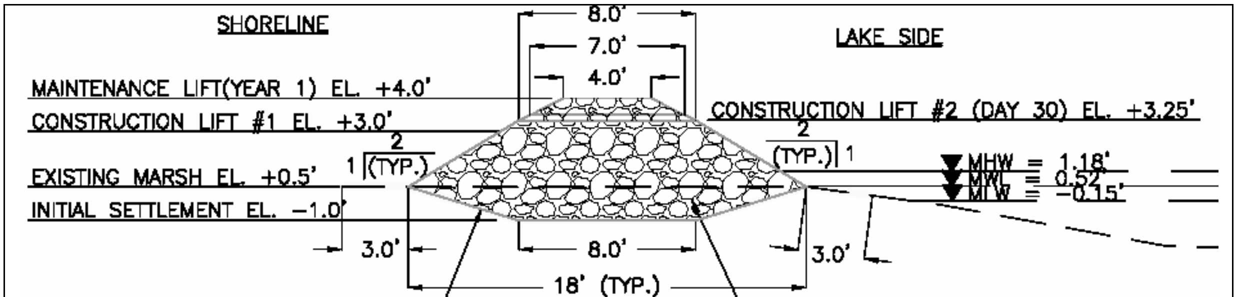

Figure 16 – Typical "Weak" Breakwater Section

7.5. Breakwater Alignment

The alignment of the rock breakwater is placed along the +0.5 ft NAVD88 contour using 1000-foot straight line segments. These straight line segments will create a more natural alignment for the rock breakwater to protect against wave energies. Construction surveying and stake out will also be more uniformly facilitated using straight line segments. The plan view for the alignment of the proposed breakwater is provided in the plans.

7.6. Flotation and Access Channels

Two barges will be aligned side by side but parallel to the shoreline during construction of the rock breakwater. One barge will support a long reach track-hoe and the other will supply the rock riprap. The minimum width for the flotation channel is therefore set at 80 feet based upon the width of two standard barges. For flotation access channels, the minimum width is set at 120 feet in order to allow an adequate turning radius for the barges.

The typical draft for barges fully loaded with stone is -8.0 below the water line. The depth of the access and flotation channels is set at -6.0 ft NAVD88 which yields a total draft of approximately 7.0 ft after adding the mean water elevation. At this depth, the barges may be limited to partial loading, however less spoil will need to be dredged and subsequently backfilled.

A 25 foot buffer between the flotation channel and the spoil stockpile was set to maintain slope stability for the temporary spoil stockpile. As discussed in Section 3.4, the minimum distance required to maintain adequate slope stability of the breakwater is set at 50 feet from the flotation channel. The alignment of the flotation channel is therefore set at 50 feet from the outside toe of the rock breakwater. The slope of the flotation channel is set at 2H:1V in order to match the slope of the breakwater. A typical section of the breakwater, flotation channel, and spoil stockpile is shown in Figure 17.

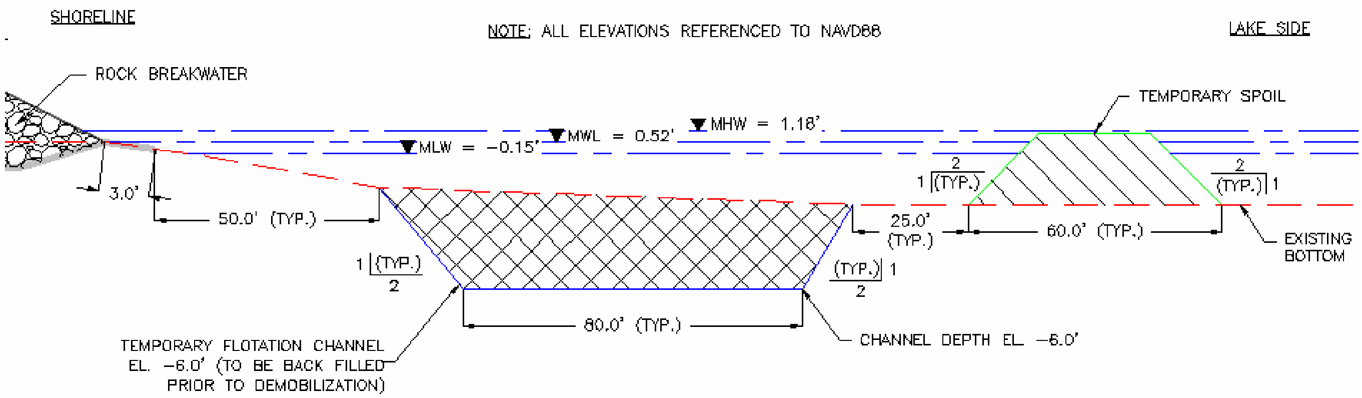

Figure 17 (NTS) – Typical Section of Flotation Channel

A total of four access routes will be strategically aligned from the lake in order to facilitate barge access to the flotation channels at the center of the corresponding reach. A typical section of the flotation channel and spoil stockpile is shown in Figure 18.

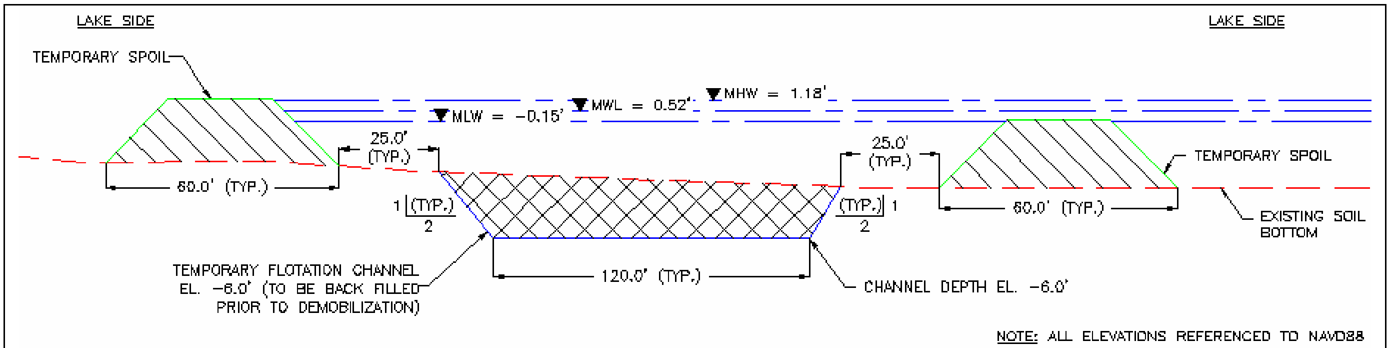

Figure 18 (NTS) – Typical Section of Access Channel

Flotation channels will not be provided along Additive Alternate #1 and around the former naval station. Instead, construction of the rock breakwaters along these two areas will be accomplished onshore using end-on-construction techniques. The locations of the alignments of the access and flotation channels are shown in the plans.

8. ADDITIVE ALTERNATE AND END-ON-CONSTRUCTION

End-on-construction does not require flotation access because all activities will be performed within the footprint of the breakwater. Equipment and materials access will be provided to the shore from flotation channels on adjacent construction reaches. Costs for construction using this technique, however, are more expensive due to the need for additional equipment and required expansion of the footprint for equipment travel. A typical section of the rock breakwater created through end-on-construction is shown in Figure 19.

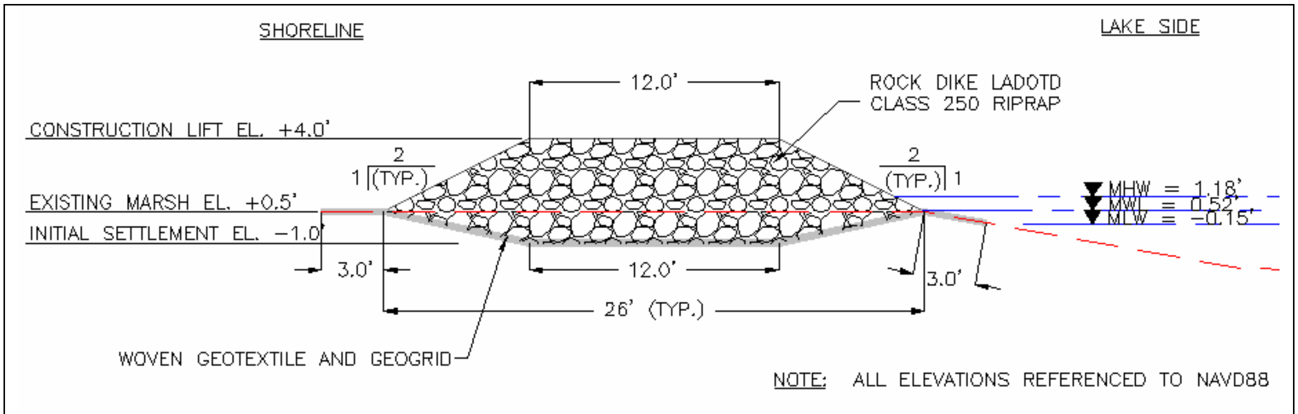

Figure 19 – Typical Section of Rock Breakwater Using End-On-Construction

Approximately 1,534 ft of rock breakwater along the former naval base will be constructed using end-on-construction in order to avoid the vast debris which exists in the area. Approximately 2,182 ft of rock breakwater along cultural resource sites SB39 and SB40 at the northeast end of the Bayou Dupre reach will also be constructed using end-on-construction as an added alternate depending upon availability of construction funds and resolution of cultural resources issues. The estimated materials quantities are provided in Attachment E. Refer to Section 10.0 for further information on the cultural resources sites.

9. SHORELINE PROTECTION STRUCTURE AT BAYOU DUPRE

As discussed in Section 6, the most cost effective shoreline protection feature at the mouth of Bayou Dupre is a back-to-back fiberglass sheet pile structure backfilled with coarse grained (sandy) material. This structure will be designed to resist the overturning and sliding moment developed from the deep water wave. A top layer of stone separated by geotextile will limit erosion of the sand layer from overtopping waves. An isometric view of the structure is shown in Figure 22. The estimated materials quantities are provided in Attachment E. The final analysis and design of the structure will now be discussed.

9.1. Wave Load Determination

The deep water wave condition was utilized in the design of the structure due to the fact that the bathymetry does not incur shoaling at the mouth of Bayou Dupre. The elevation of the existing mud line along the alignment ranges from -2 to -8 ft NAVD88. The pressure distribution of the deep water wave was developed using the Miche-Rundgren formula for non-breaking waves against vertical walls as shown in Attachment G. Impulsive forces from breaking waves were not incorporated into the design due to the low probability of an entire wave assaulting the entire structure simultaneously.

The structure will be designed to remain fully saturated by providing weep holes at elevation -2.0 ft NAVD88. Due to full saturation, the overall force acting against the structure will be reduced by an amount equal to the force caused by the hydrostatic pressure. The resultant force and overturning moment for the deep water wave minus the hydrostatic portion of the pressure distribution are calculated to be 1,109 lb/ft and 5,461 ft-lbs, respectively.

9.2. External Stability Analysis of Soil Mass

The design criteria used to evaluate the soil mass contained within the proposed back-to-back fiberglass sheet pile wall is based on methodologies developed for designing Mechanically Stabilized Earth Walls (MSEW), which are used to retain soil. MSE Walls generally consist of a granular backfill material, reinforcing elements within the backfill, and a facing. These systems are usually constructed in fill applications by placing alternating layers of soil and reinforcing elements. The weight of the reinforced soil structure is then used to resist overturning and sliding forces developed from the retained soil (Figure 20).

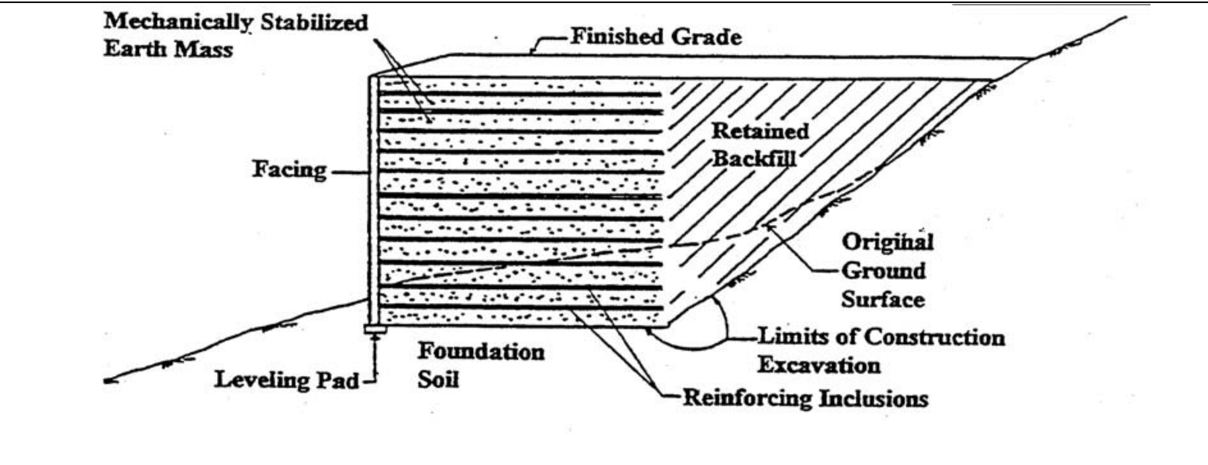

Figure 20 - Mechanically Stabilized Earth Wall System (Figure 6-1, NHI Course 13236)

The proposed back-to-back fiberglass sheet pile structure will be backfilled with a granular material to elevation 0.00 ft NAVD88. A geotextile fabric will be specified to cover the granular backfill. A rock layer will then be placed from elevation 0.0 ft NAVD88 to elevation 2.5 ft NAVD88. Therefore, the granular material and rock will be contained within the back-to-back fiberglass sheet pile structure. The buoyant unit weight and soil friction angle, phi (ø) parameters of both materials were used to determine the resisting soil mass weight at a lake bottom elevation of -5.0 ft NAVD88 and -8.0 ft NAVD88. A silty sand backfill material with a unit weight of 115 PCF and a phi angle, ø, of 20 degrees were used for design. A top of wall elevation of +2.5 ft NAVD88 was also used for design. The geotechnical parameters from Boring #3 were used to determine the foundation soil parameters. Figure 21 shown below indicates the design parameters specified above. The soil mass area consists of the rock and sand layers.

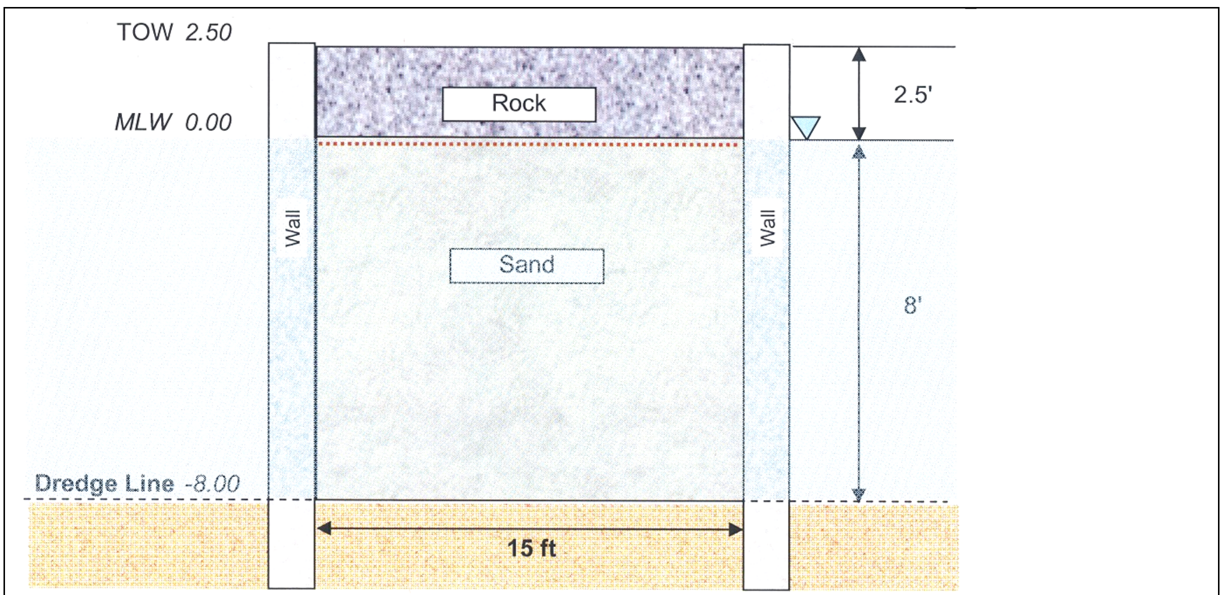

Figure 21 - Back-to-Back Fiberglass Sheeting and Soil Mass

In an effort to simplify the design, the shear resistance of the fiberglass sheeting was neglected in the overturning and sliding analyses. Several variations on the width of the soil mass were analyzed in order to determine the most optimum width of the structure. A wall width of 15 feet resulted in a $F.S._{overturning} = 7.7$, and a $F.S._{sliding} = 1.3$ for a lake bottom elevation of -8.0 NAVD88. A $F.S._{overturning} = 10.0$ and a $F.S._{sliding} = 1.9$ were determined for a lake bottom depth of -5.0 ft NAVD88. The hydrostatic force on the lake side was conservatively used in the analyses for evaluating the overturning and sliding safety factors. However, the Wave Resultant Force used in the sheet pile calculations was determined neglecting the hydrostatic force.

Based on these analyses, the soil mass weight will resist the overturning and sliding moments produced from the design wave force. The external stability analyses for each lake bottom elevation are shown in Appendix F.

9.3. Fiberglass Sheet Pile Wall

The minimum size of composite fiberglass sheet pile was determined by evaluating the deflection and bending moment caused by the maximum applied load. Three alternatives were evaluated for the maximum loading condition; soil load with wave force, soil load without wave force, and post-primary consolidated soil and upper rock layer loads without wave force. The wave load used in the design of the soil mass was also utilized in these calculations. It was assumed that the sand mass will absorb all of the wave energy, therefore the sheet pile wall on either side can be designed as a single cantilever wall. Also, due to the installment of weep holes, the hydrostatic pressure can be neglected. The weep holes shall be protected from plugging by overlaying each hole with geotextile fabric during the backfilling process.

The maximum deflection and bending moment were calculated using the SPW911 V2.0 program created by Pilebuck, Inc. The calculations and final selection are shown in Appendix H. The optimum fiberglass sheet pile was evaluated to contain the following minimum specifications:

Modulus of Elasticity	2.8×10^6 psi
Moment of Inertia	182.47 in^2/ft
Section Modulus	26.06 in^3/ft
Working Stress	12,500 psi
Allowable Bending Moment	27,145 ft-lbs/ft
Width	18 in
Thickness	0.30 in
Depth	14 in

In order to maintain a continuous span of sheet piles along the alignment, a combination of stainless steel walers and composite fiberglass tie rods were selected based on allowable loading, flexure and shear as shown in Appendix H. The optimum location for placement of the waler and tie rods on the sheet pile span occurs at elevation 0 ft NAVD88. The optimum spacing for the tie rods occurs along 4 foot intervals.

The back-to-back fiberglass sheet pile structure will tie the existing USACE rock breakwaters to the rock breakwaters proposed for this project. The existing USACE rock breakwaters will be extended to the structure by the addition of stone using the original geometry of the breakwaters. The proposed breakwaters will simply be tied in along the alignment during construction.

9.4. Scour Protection

The toe of the back-to-back fiberglass sheet pile structure will be protected against wave scour by the use of a rock berm. The dimensions of the typical cross section for the rock berm were determined from the Markle Equation (1989) in Table VI-5-45 of the USACE CEM. The design wave height and maximum mud line depth of -8.0 ft NAVD88 were utilized in the calculations. The results of these calculations showed that no scour protection is warranted for the given design conditions. In order to remain conservative, a small berm is proposed to be constructed along the outside toe of the structure with the following dimensions; crest height 2 ft above the mud line with a 2:1 side slope. A typical isometric view of the proposed back-to-back fiberglass sheet pile structure is shown in Figure 22.

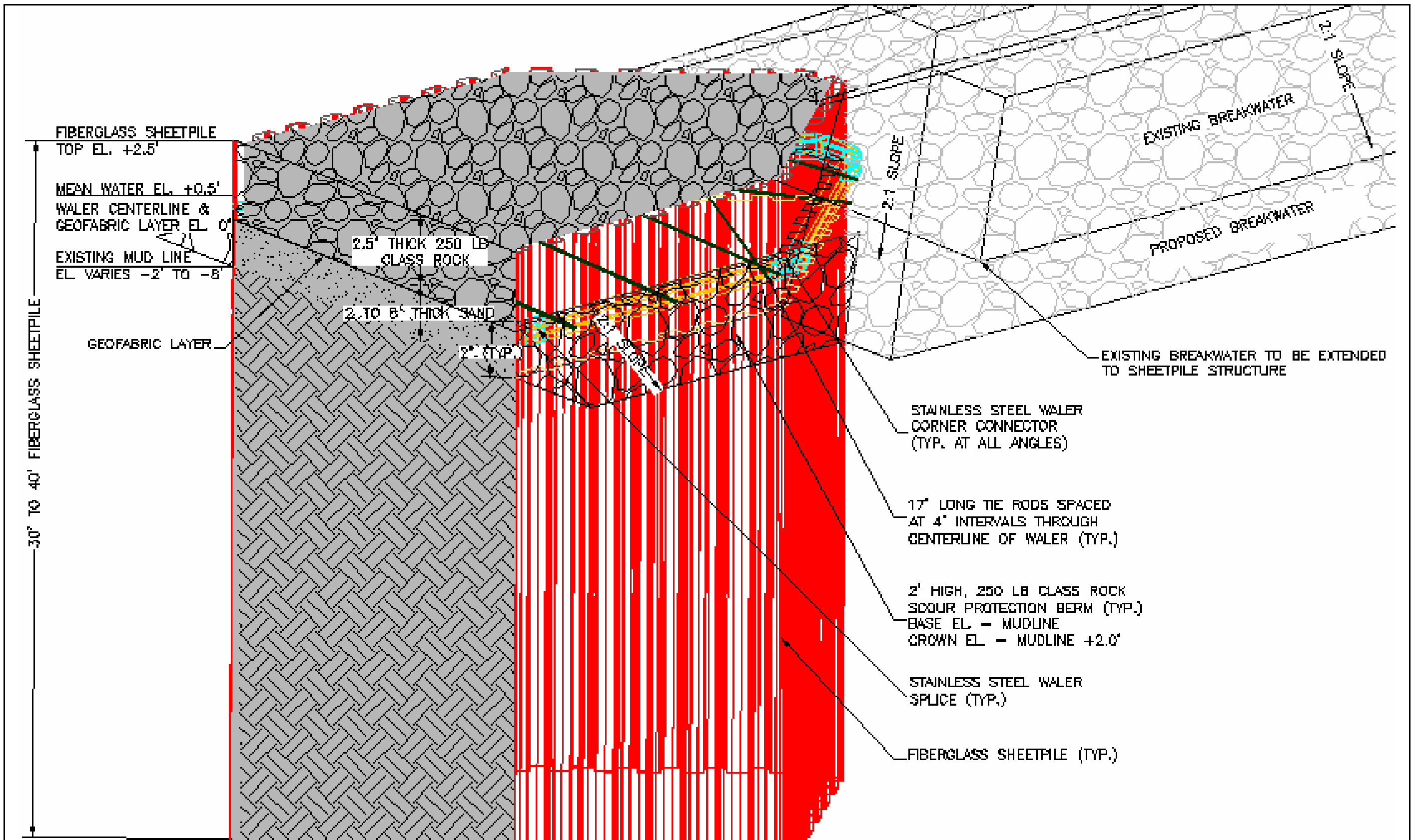

Figure 22 – Isometric View of Back-to-Back Fiberglass Sheet Pile Structure

10. CULTURAL RESOURCES

Louisiana has a long and rich history and cultural resources are commonly discovered within project footprints. State of Louisiana State Historic Preservation Office (SHPO) files revealed sites of potential interest within the project areas. A field visit conducted by EPA, DNR, SHPO, Chitimacha Tribe of Louisiana and the Mississippi Band of Choctaw Indians on April 23, 2003 confirmed the presence of cultural resources potentially within the project footprint.

LDNR contracted for a Phase I archeological survey conducted by C&C Technologies. This survey was conducted from February 26, 2004 through March 6, 2004. This survey was performed in accordance with SHPO Phase I guidelines, which included a terrestrial and submerged cultural survey. C&C Technologies surveyed the entire project footprint, which extended 15 feet inland from the waters edge and into Lake Borgne to either a 6 ft depth or a distance 1000 ft from the shoreline. A total of 399 acres were investigated as part of this survey.

A final report was produced locating the sites and identification of new sites. There were a total of 4 cultural sites. Three of the sites were previously identified from work by others. The new site located from this investigation and a previously identified site was determined to be eligible. These two sites are located at the end of the southern Bayou Dupre segment. At this time, discussions with the SHPO and the tribes are ongoing. Unless written concurrences from the tribes and SHPO is received stating that the project will have no adverse impacts, total avoidance of the sites in question will preclude shoreline protection of these areas and a buffer distance of 500’ away from these sites will be maintained during construction. In order to proceed with design efforts and construction, assuming funds are approved by the CWPPRA Task Force, the project is designed as two separate areas, base bid and an additive alternate. The additive alternate is the area with cultural sites and will be included provided concurrence as outlined above is received from the tribes and SHPO.

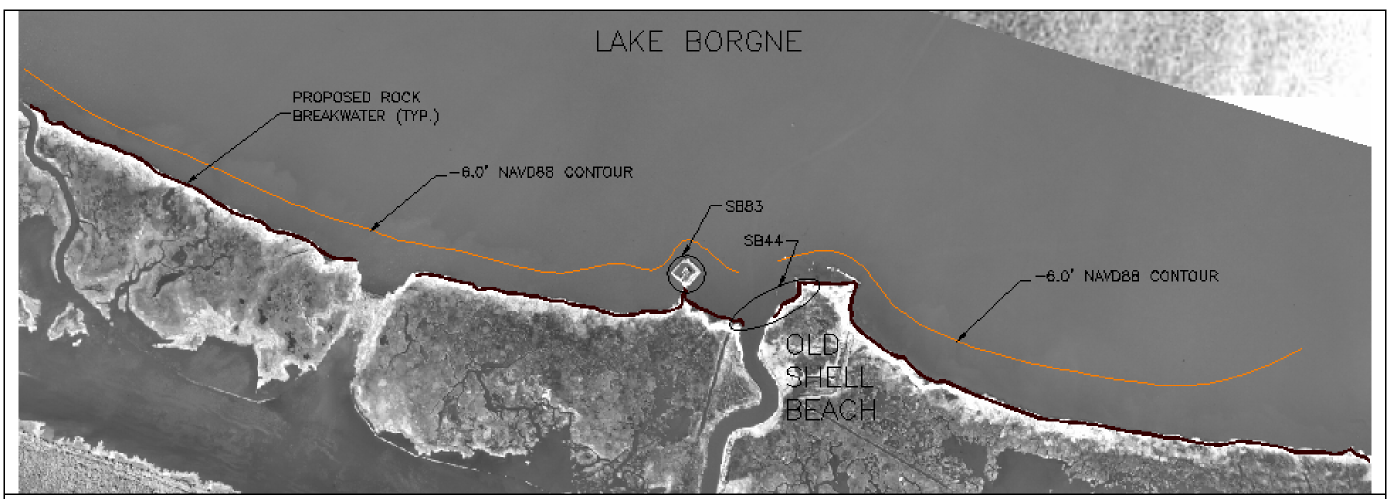

Figure 23 – Cultural Resource Sites at Shell Beach

Z

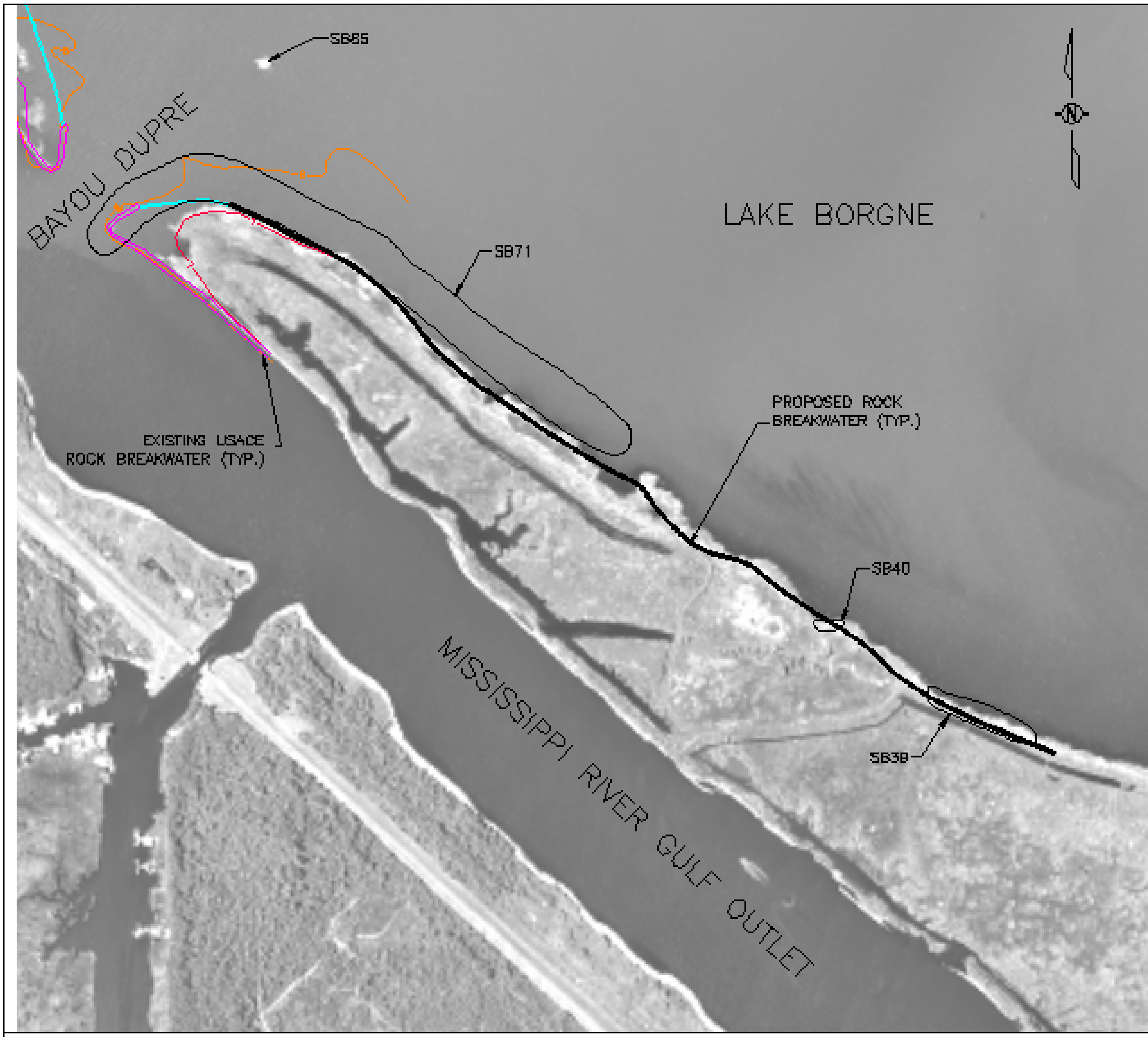

Figure 24 – Cultural Resource Sites at Bayou Dupre

11. REAL ESTATE AND OYSTER LEASES

The Louisiana Department of Natural Resources, Office of Coastal Restoration Land Rights Section (LDNR LR) coordinated the land rights. The LDNR LR Section identified 26 landowners within 14 tracts. LDNR has signed contracts with 25 of the 26 landowners. Attempts to contact the remaining landowner have not been successful.

There are 6 oyster leases in the project area which encompasses 338 acres (Figures 25 and 26). The leases have a lease value of $91,200 and a standing crop value of $147,959 for a total value of $239,159. This estimate will be refined prior to the 95 Percent Design review. The state is currently evaluating its oyster lease policy and is not currently negotiating with lease holders. We hope to have a resolution in short order.

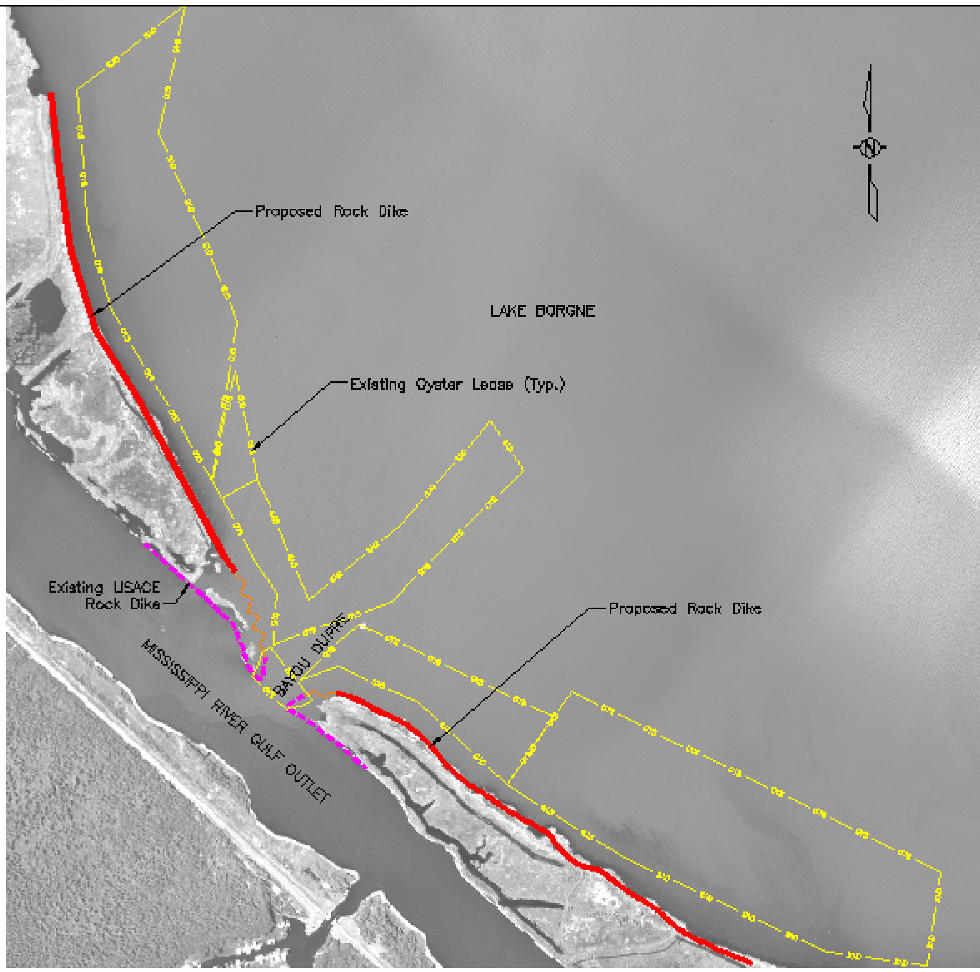

Figure 25 – Oyster Leases at Bayou Dupre

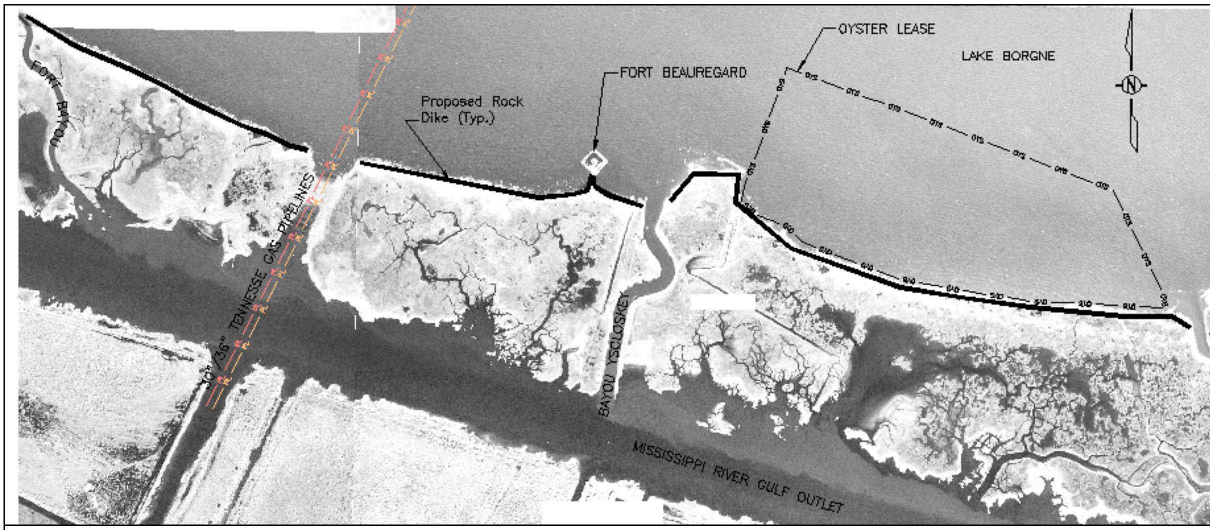

Figure 26 – Oyster Lease at Shell Beach

12. REFERENCES

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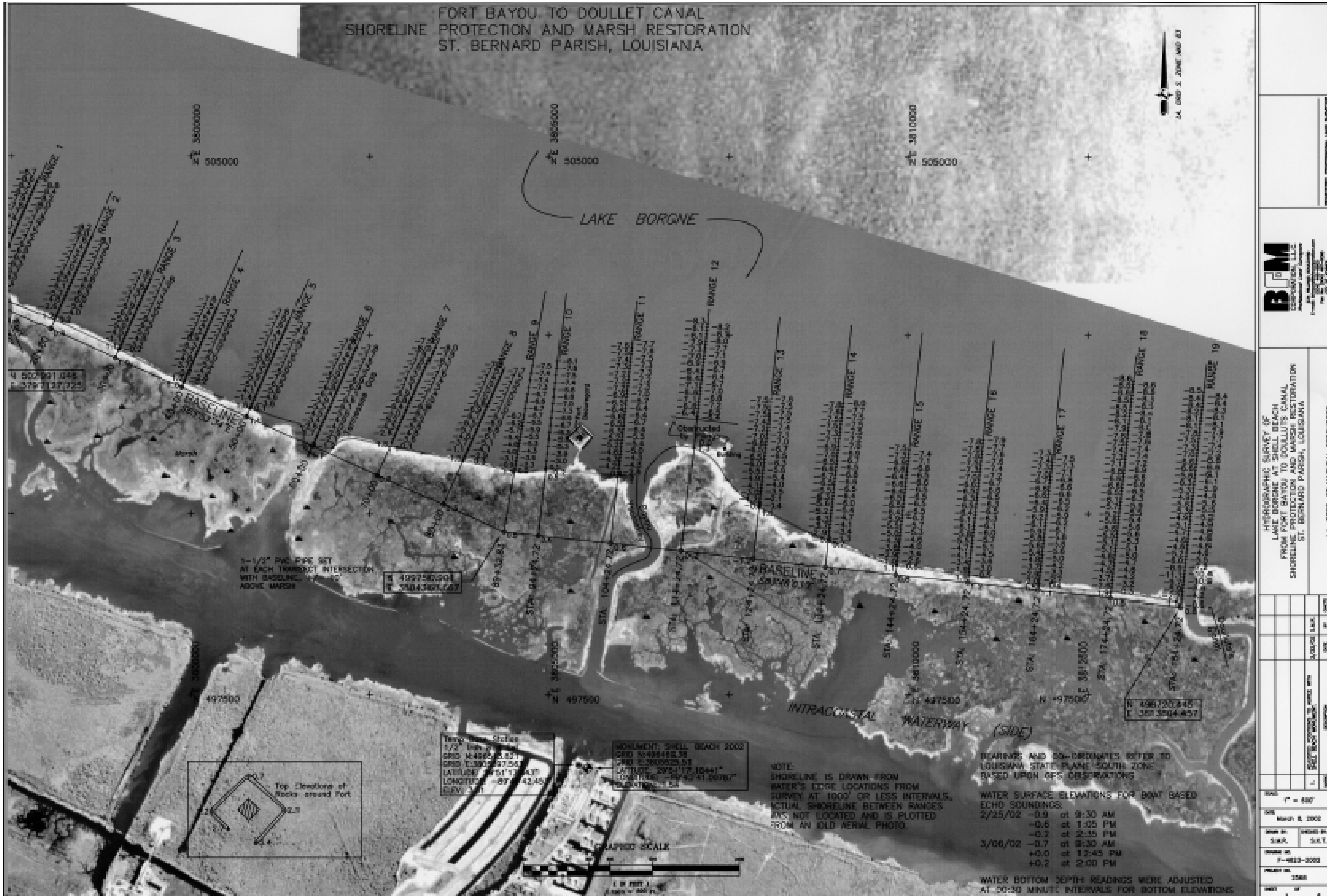
NHI Course No. 13236 – Module 6, Earth Retaining Structures, May 1998, U. S. Department of Transportation; Federal Highway Administration, National Highway Institute.

Appendix A

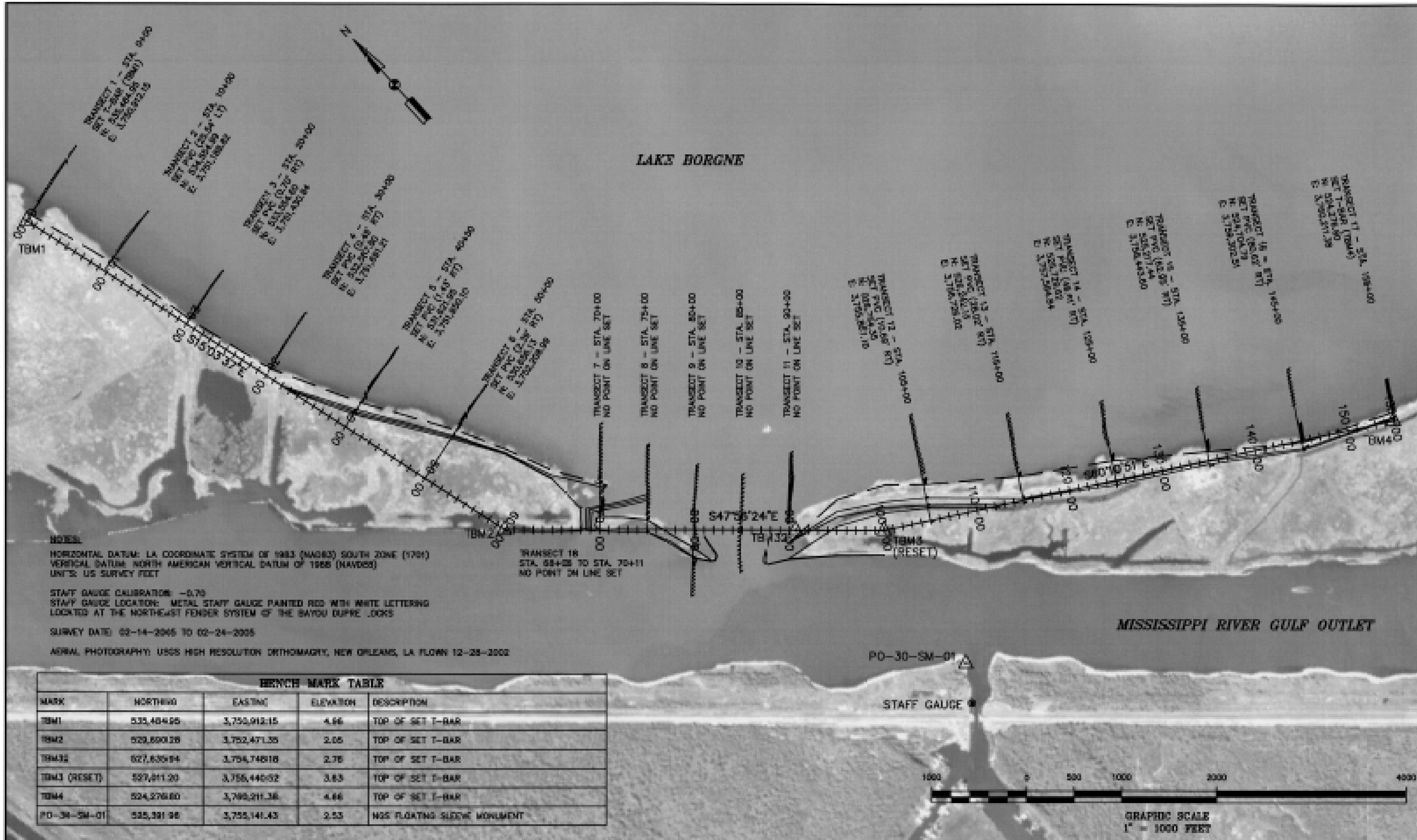
Topographic, Bathymetric and Magnetometer Survey – Lake Borgne at Shell Beach



FORT BAYOU TO DOULLET CANAL
SHORELINE PROTECTION AND MARSH RESTORATION
ST. BERNARD PARISH, LOUISIANA
LAKE BORGNE
INTRACOASTAL WATERWAY (SIDE)
BASELINE
NOTE:
SHORELINE IS DRAWN FROM
WATER'S EDGE LOCATIONS FROM
SURVEY AT 1000' OR LESS INTERVALS.
ACTUAL SHORELINE BETWEEN RANGES
WAS NOT LOCATED AND IS PLOTTED
FROM AN OLD AERIAL PHOTO.
BEARINGS AND CO-ORDINATES REFER TO
LOUISIANA STATE PLANE SOUTH ZONE
BASED UPON GPS OBSERVATIONS
WATER SURFACE ELEVATIONS FOR BOAT BASED
ECHO SOUNDINGS:
2/25/02 -0.9 at 9:30 AM
-0.6 at 1:05 PM
-0.2 at 2:35 PM
3/06/02 -0.7 at 9:30 AM
+0.0 at 12:45 PM
+0.2 at 2:00 PM
WATER BOTTOM DEPTH READINGS WERE ADJUSTED
AT 30:30 MINUTE INTERVALS FOR BOTTOM ELEVATIONS
1-1/2" PVC PIPE SET
AT EACH TRANSECT INTERSECTION
WITH BASELINE
ABOVE MARSH
Top Elevations of Rocks around Fort
GRAPHIC SCALE
HYDROGRAPHIC SURVEY OF
LAKE BORGNE AT SHELL BEACH
FROM FORT BAYOU TO DOULLUT'S CANAL
SHORELINE PROTECTION AND MARSH RESTORATION
ST. BERNARD PARISH, LOUISIANA
LA. DEPT. OF NATURAL RESOURCES
March 8, 2002

Appendix B

Topographic and Bathymetric Survey – Lake Borgne at Bayou Dupre



NOTES:

HORIZONTAL DATUM: LA COORDINATE SYSTEM OF 1983 (NAD83) SOUTH ZONE (1701)
VERTICAL DATUM: NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88)
UNITS: US SURVEY FEET

STAFF GAUGE CALIBRATION: -0.70
STAFF GAUGE LOCATION: METAL STAFF GAUGE PAINTED RED WITH WHITE LETTERING LOCATED AT THE NORTHEAST FENDER SYSTEM OF THE BAYOU DUPRE LOCKS

SURVEY DATE: 02-14-2005 TO 02-24-2005

AERIAL PHOTOGRAPHY: USGS HIGH RESOLUTION ORTHOIMAGERY, NEW ORLEANS, LA FLOWN 12-28-2002

BENCH MARK TABLE

MARK	NORTHING	EASTING	ELEVATION	DESCRIPTION
TBM1	535,464.95	3,750,912.15	4.66	TOP OF SET T-BAR
TBM2	529,690.28	3,752,471.35	2.05	TOP OF SET T-BAR
TBM32	527,630.64	3,754,748.18	2.76	TOP OF SET T-BAR
TBM3 (RESET)	527,011.20	3,755,440.52	3.63	TOP OF SET T-BAR
TBM4	524,276.60	3,760,211.36	4.86	TOP OF SET T-BAR
PO-30-SM-01	525,391.98	3,755,141.43	2.53	NGS FLOATING SLEEVE MONUMENT

PRELIMINARY

LOUISIANA DEPARTMENT OF NATURAL RESOURCES
COASTAL RESTORATION DIVISION
617 NORTH 3RD STREET
BATON ROUGE, LOUISIANA 70802

LAKE BORGNE SHORELINE PROTECTION
CWPPRA PROJECT PO-30
ST. BERNARD PARISH, LA
TOPOGRAPHIC & BATHYMETRIC SURVEY

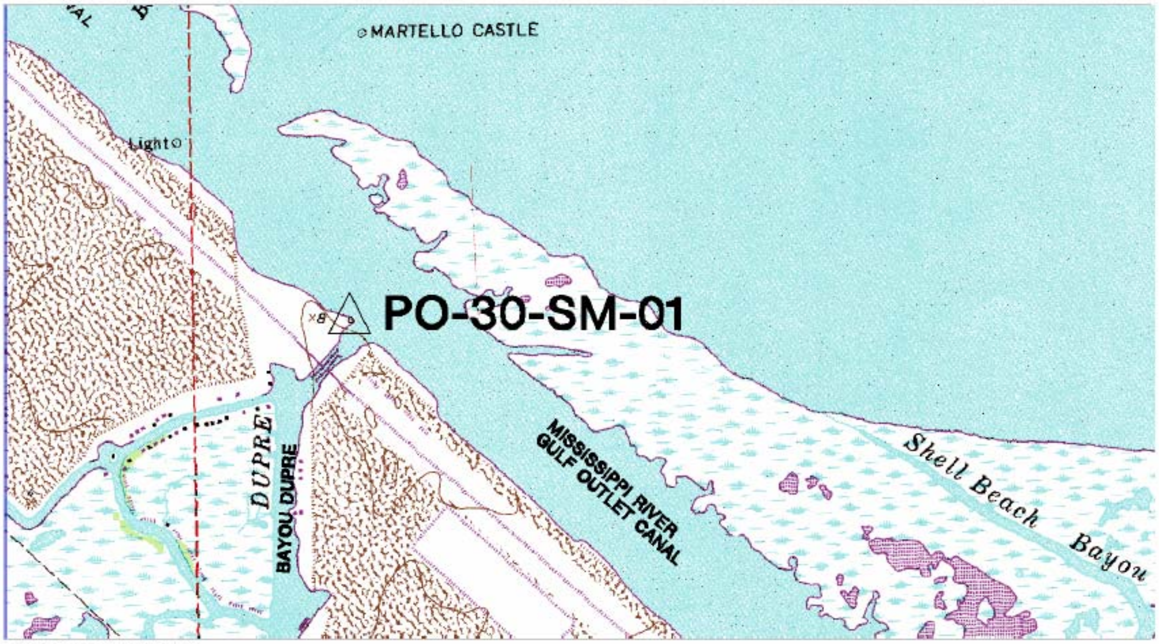
SURVEY PLAN VIEW

STATE PROJECT: PO-30
SCALE: 1"=1000'
SHEET NUMBER: 1
DESIGNED: R.E.
DRAWN: R.E.
DATE: 03-21-2005

SIGMA CONSULTING GROUP, INC.
10305 AIRLINE HWY.
BATON ROUGE, LA 70816
(225) 298-0800

Appendix C

LDNR Secondary Monument "PO-30-SM-01" Data Sheet



VICINITY MAP Scale: 1" = 2000' Reproduced from USC&GS "Martello Castle" Quadrangle

Station Name: "PO-30-SM-01"

Location: The monument stamped "PO-30-SM-01" is located near Shell Beach, Louisiana. From the intersection of Paris Road and LA Hwy. 39 (Judge Perez Road) in Chalmette proceed east on LA Hwy. 39 for 8.1 miles to the intersection of LA Hwy. 39 and LA Hwy. 46 near St. Bernard High School. Proceed east on LA Hwy. 46 for 6.3 miles to a levee on the left. Follow the levee for approximately 7.7 miles to the Bayou Dupre Floodgates. The monument is located at the intersection of the west bank of Bayou Dupre and southern bank of the Mississippi River Gulf Outlet Canal. It is approximately 800 ft. northeast of the northern wing wall of the Bayou Dupre flood control structure behind the rip-rap lined bank. Access across the flood control structure should be coordinated with the St. Bernard Parish Levee District.

Monument Description: NGS style floating sleeve monument; datum point set on 9/16" stainless steel sectional rods driven 28 feet to refusal, set in sand filled 6" PVC pipe with access cover set in concrete, flush with ground.

Stamping: PO-30-SM-01

Installation Date: 2003 **Date of Survey:** Nov. 19-21, 2003

Monument Established By: Sigma Consulting Group, Inc.

For: *Louisiana Department of Natural Resources, CRD*

Adjusted NAD 83 Geodetic Position
Lat. 29°56' 10.33674" N
Long. 89°50' 08.86486" W

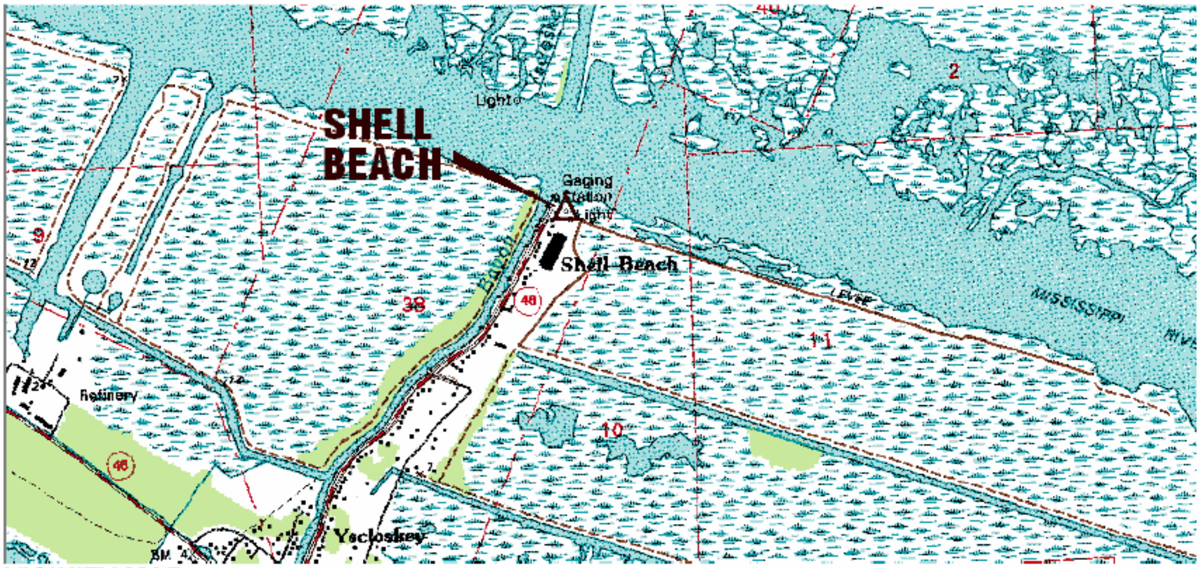
Adjusted NAD 83 Datum LSZ (1702) Feet
N= 525,391.96
E= 3,755,141.43

Adjusted NAVD88 Height
Elevation = 2.53 feet (0.772 mtrs)
Geoid99 Height = -26.109 mtrs.
Ellipsoid Height = -25.338 mtrs.

Adjusted Position Established for Louisiana Department of Natural Resources, Coastal Restoration Division

Appendix D

LDNR Secondary Monument "SHELL BEACH-2002" Data Sheet



VICINITY MAP Scale 1" = 2000' Reproduced from USC&G "Ysclosky" Quadrangle

Station Name: **"SHELL BEACH 2002"**

Monument Location: From the intersection of LA Hwy 46 & LA Hwy 300 in Reggio at the flashing signal light at the "The Junction Store", proceed east on LA Hwy 46 approximately 4.6 miles to a drawbridge. Proceed north across drawbridge over Bayou La Loutre to the intersection of LA Hwy 46 & LA Hwy 624, then head west 0.2 miles to a road that turns north along Bayou Ysclosky. Proceed north along winding road on the east side of Bayou Ysclosky for 1.2 miles to the end of the road at the Intracoastal Waterway. Mark is on the right (east side) of the road on the south edge of a shell parking area. 175 feet east of centerline of road; 75 feet Southeast of wood pole with meter; located at south edge of shell parking area.

Monument Description: Stainless steel rod driven to point of refusal (72' deep) within a sleeve and protective cover set in concrete and stamped "Shell Beach 2002".

Date: March 2002

Monument Established by: BFM Corporation

NAD 83 Geodetic Position

Lat. 29°51'17.18441"
Long. 89°40'41.00787"

La. State Plane South Zone(NAD 83)
N= 496,469.38
E= 3,805,525.51

NAVD 88(Feet)/Geoid 99
Elevation= 1.54feet/0.469meters

Ellipsoid Height = -25.400 meters
Geoid99 Height = -25.868 meters

Appendix E

Cost Estimates

PO30 (Lake Borgne) Cost Estimate for Steel Sheet pile - Top Elevation +2.5 ft NAVD88

					Steel Sheet piles				Battered Timber Piles	
Location	Reach	Lift	Year	Linear Length (ft)	Crown Elevation NAVD88	# of Sheet pile Rows	CADD Area (Ft^2)	Cost $\$34/Yd^2$ ($)	# of 35' Long Piles (Each)	Cost $455/Pile ($)
Bayou Dupre	1	1	1	6,643	2.5	1	132860	$4,517,240		
	West	1	1	1,163	2.5	1	31,413	1,068,042	194	88,194
	East	1	1	439	2.5	1	10,975	373,150	73	33,291
	2	1	1	6,418			128,360	4,364,240		
Shell Beach	3	1	1	7,864	2.5	1	157,280	5,347,520		
	4	1	1	9217	2.5	1	184,340	6,267,560		
							645,228	21,937,752	267	121,485

	Total Cost
Mob/Demob	1,000,000
Total Cost	23,059,237
Total Cost +15%	26,518,123

PO30 (Lake Borgne) Cost Estimate Segmented Concrete Panels - Crown Elevation +2.5 ft NAVD88

Location	Reach	Lift	Year	Linear Length (ft)	Concrete Panels Crown Elevation NAVD88	Concrete Panels Cost $350/LF ($)
Bayou Dupre	1	1	1	6,643	2.5	2,325,050
	West	1	1	1,163	2.5	407,050
	East	1	1	439	2.5	153,650
	2	1	1	6,418	2.5	2,246,300
Shell Beach	3	1	1	7,864	2.5	2,752,400
	4	1	1	9217	2.5	3,225,950
						11,110,400

	Total Cost
Mob/Demob	1,000,000
Cost	12,110,400
+15%	13,926,960

PO30 (Lake Borgne) Cost Estimate for Rock Breakwater - Outside Toe Elevation +0.5 ft NAVD88 - Construction Lift at +4 ft NAVD88 and 1 Maintenance Lift at +4 ft NAVD88 (2 @ Mouth of Bayou Dupre)

Location	Reach	Lift	Year	Linear Length (ft)	Rock Dike & Scour Protection: Crown Elevation NAVD88	Crown Width (ft)	Side Slopes (ft/ft)	CADD Volume (Yd^3)	Elastic Settlement Multiplier	Voids Added (%)	Total Volume (Yd^3)	Total Weight (Tons)	Cost \$25/Ton (\$)	Flotation: Bottom Elevation NAVD88	Bottom Width (ft)	Side Slopes (ft/ft)	CADD Volume (Yd^3)	Cost \$2/Yd^3 (\$)	Geotextile/grid: CADD Area (Yd^2)	Cost \$7/Yd^2 (\$)
Bayou Dupre	1	1	1	6,643	4	4	2:1	10,349	1.5	10	17,076	35,731	893,280	-6	80	2:1	64,335	128,670	20,081	140,567
			5		1	4	2:1													
		2			4	4	2:1	7,343	1.0	10	8,077	16,902	422,544	-6	80	2:1	64,335	128,670		
	West	1	1	1,163	4	4	2:1	10,169	1.5	10	14,733	30,829	770,720	-6	80	2:1	908	1,816	15,253	106,771
			5		1	4	2:1													
		2			4	4	2:1	6,101	1.0	10	6,711	14,043	351,074	-6	80	2:1	908	1,816		
			10		1	4	2:1													
		3			4	4	2:1	6,101	1.0	10	6,711	14,043	351,074	-6	80	2:1	908	1,816		
	East	1	1	439	4	4	2:1	2,685	1.5	10	4,430	9,270	231,757	-6	80	2:1	1,209	2,418	5,236	36,652
			5		1	4	2:1													
		2			4	4	2:1	1,611	1.0	10	1,772	3,708	92,703	-6	80	2:1	1,209	2,418		
			10		1	4	2:1													
		3			4	4	2:1	1,611	1.0	10	1,772	3,708	92,703	-6	80	2:1	1,209	2,418		
	2	1	1	6,418	4	4	2:1	8,695	1.5	10	14,347	30,021	750,514	-6	80	2:1	62,156	124,312	13,763	96,343
			5		1	4	2:1													
		2			4	4	2:1	5,250	1.0	10	5,775	12,084	302,105	-6	80	2:1	62,156	124,312		
Shell Beach	3	1	1	7,864	4	4	2:1	11,088	1.5	10	18,295	38,283	957,068	-6	80	2:1	92,350	184,700	22,529	157,703
			5		1	4	2:1													
		2			4	4	2:1	9,777	1.0	10	10,755	22,504	562,605	-6	80	2:1	92,350	184,700		
	4	1	1	9217	4	4	2:1	12,318	1.5	10	20,325	42,529	1,063,236	-6	80	2:1	83,979	167,958	24,254	169,776
			5		1	4	2:1													
		2			4	4	2:1	8,622	1.0	10	9,484	19,846	496,142	-6	80	2:1	121,896	243,792		
								101,720			140,263	293,501	7,337,526				649,908	1,299,816	101,116	707,811

Task	Initial Construction	Maintenance Lift 1st	Maintenance Lift 2nd	Total Cost
Mob/Demob	1,000,000	1,000,000	1,000,000	2,000,000
Total Cost	6,984,261	3,912,881	1,448,011	10,897,142
Total Cost +15%	8,031,900	4,499,813	1,665,213	14,196,927

PO30 (Lake Borgne) Cost Estimate for Rock Breakwater and Fiberglass Sheetpile - Outside Toe Elevation +0.5 ft NAVD88 - Construction Lift at +3, +3.25 and +4 ft NAVD88 and 1 Maintenance Lift at +4 ft NAVD88

					Rock Dike & Scour Protection									Flotation				
Location	Reach	Lift	Year	Linear Length (ft)	Crown Elevation NAVD88	Crown Width (ft)	Side Slopes (ft/ft)	CADD Volume (Yd^3)	Elastic Settlement Multiplier	Waste Added (%)	Total Volume (Yd^3)	Total Weight (Tons)	Cost $25/Ton ($)	Bottom Elevation NAVD88	Bottom Width (ft)	Side Slopes (ft/ft)	CADD Volume (Yd^3)	Cost $2/$Yd^3$ ($)
Bayou Dupre	1	1	0	6,643	3	8	2:1	8,873	1.5	10	14,640	30,635	765,879	-6	80	2:1	64,335	128,670
			0.1		2	8	2:1											
		2			3.25	7	2:1	2,905	1.0	10	3,196	6,687	167,165					
			1		1.7	7	2:1											
		3			4	4	2:1	4,841	1.0	10	5,325	11,143	278,569	-6	80	2:1	58,391	116,782
	1 Scour	1	0	1,154	2 AML	0	2:1	1,620	1.5	10	2,673	5,593	139,831					
	1 Fill	1	0	1,154	2.5	15	0	1,592	1.5	10	2,627	5,497	137,414					
	2 Scour	1	0	439	2 AML	0	2:1	630	1.5	10	1,040	2,175	54,379					
	2 Fill	1	0	439	2.5	15	0	615	1.5	10	1,015	2,123	53,084					
	2	1	0	4,418	4	4	2:1	5,003	1.5	10	8,255	17,273	431,837	-6	80	2:1	45,224	90,448
			20		3	4	2:1											
	Alt. Add.	1	0	2,000	4	12	2:1	6,177	1.0	10	6,795	14,218	355,448	-6	80	2:1	37,561	75,122
			20		3	12	2:1											
Shell Beach	3	1	0	7,864	3 or 4	8	2:1	9,947	1.5	10	16,413	34,343	858,582	-6	80	2:1	87,923	175,846
			0.1		2 or N/A	8	2:1											
		2			3.25 or N/A	7	2:1	1,867	1.0	10	2,054	4,297	107,434					
			1		1.7 or N/A	7	2:1											
		3			4 or N/A	4	2:1	3,113	1.0	10	3,424	7,165	179,134	-6	80	2:1	44,410	88,820
	4	1	0	9217	4	4	2:1	13,981	1.5	10	23,069	48,271	1,206,779	-6	80	2:1	83,979	167,958
			20		3	4	2:1											
								61,164			90,524	189,421	4,735,534				421,823	843,646

PO30 (Lake Borgne) Cost Estimate for Rock Breakwater and Fiberglass Sheet Pile - Outside Toe Elevation +0.5 ft NAVD88 - Construction Lift at +3, +3.25 and +4 ft NAVD88 and 1 Maintenance Lift at +4 ft NAVD88

CADD Area (Yd^2)	Cost $7/$Yd^2$ ($)	# Plates @ 1000' Intervals	Cost @ $1K/Plate ($)	# of Plates @ 1000' Intervals	Cost $2000 Ea. ($)	Total Length (Ft)	Cost $15/LF ($)	Length of Sheet Pile Alignment	Cost $35/LF ($)	# of Splices @ 4' Intervals	Cost $75/Each ($)	Waler Splice # of Splices @ 24' Intervals	Waler Splice Cost $60 Ea. ($)	SS Washers # of Splices @ 24' Intervals	SS Washers Cost $10 Ea. ($)	Sand Fill CADD Volume (Yd^3)	Sand Fill Cost $8/$Yd^3$ ($)
17,462	122,235	7	7,000	7	14,000												
1044	7,310			2	4,000	50,049	750,735	2320	81,200	286	21,469	48	2,863	48	477	3,664	29,312
1907	13,351																
447	3,128			2	4,000	17,487	262,305	870	30,450	210	15,750	35	2,100	35	350	986	7,888
739	5,173																
11,879	83,151	5	5,000	5	10,000												
8,072	56,506	2	2,000	2	4,000												
22,529	157,700	9	9,000	6	12,000												
24,254	169,776	8	8,000	8	16,000												
88,333	618,330	31	31,000	32	64,000	67,536	1,013,040	3,190	111,650	496	37,219	83	4,963	83	827	4,650	37,200

PREFERRED ALTERNATIVE

Task	Initial Construction 1st	Initial Construction 2nd	Maintenance Lift	Total Cost	Alternate Additive	Total Cost + Alternate
Mob/Demob	1,000,000	0	1,000,000	2,000,000	0	2,000,000
Total Cost 7,078,429	274,599	1,663,305	9,016,333	493,076	9,509,409	
Total Cost +15%	8,140,193	315,789	1,912,801	10,368,783	567,037	10,935,820

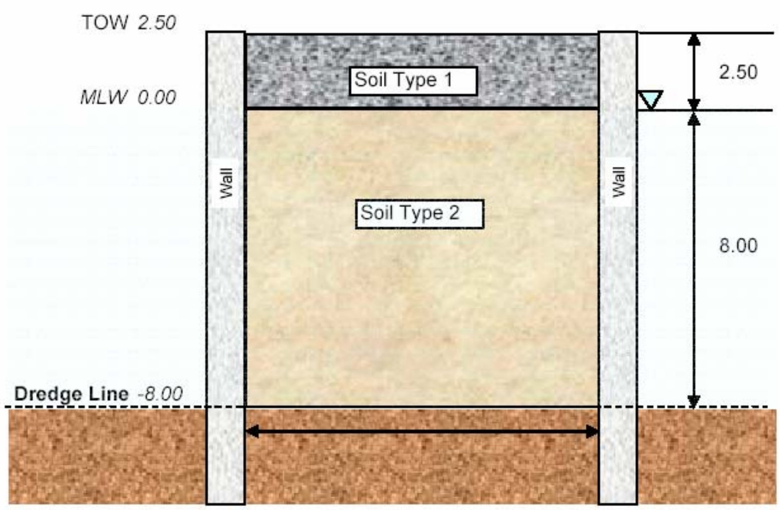
Appendix F

Stability Analysis of Soil Mass

EXTERNAL STABILITY ANALYSES OF SOIL MASS

Back-to-Back Fiberglass Sheetpile Walls- Initial Static Condition

Case @ -8.00



Soil Data Input

Soil Type 1- Unit Weight	135	lb/cu.ft.
ST1- cohesion, C	0	psf
ST1- phi angle	40	degrees
Soil Type 2- Unit Weight	115	lb/cu.ft.
Soil Type 2- Bouyant Unit Wt.	51	lb/cu.ft.
ST2- cohesion, C	0	psf
ST2- phi angle	20	degrees
Tan (phi)	0.36397	

Wall Geometry

Top of Wall (TOW)	2.50	
MLW	0.00	
Dredge Line	-8.00	
Wall Width, W	15.00	ft.

Volumes per unit length (1 foot)

ST1- Volume	37.5	cu. Yds.
ST2- Volume	120.0	cu. Yds.

Resisting Soil Mass Weight (water level at MLW)

ST1- Weight	5062.5	lbs./ft
ST2- Weight	6120	lbs./ft
Total Soil Mass Weight	***11182.5***	lbs./ft

Resisting Soil Mass Weight (water level at TOW)

Total Soil Mass Weight	***8782.5***	lbs./ft

Assumptions:

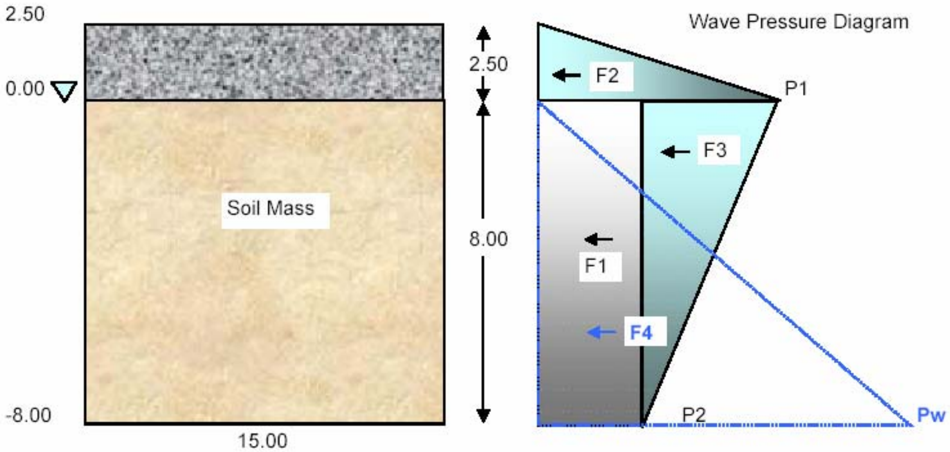
The soil mass is confined by the back-to-back walls and the woven geosynthetic fabric. The external stability analyses is determined from the NHI Couse No. 13236- Module 6, Earth Retaining Strucrures, May 1998, Figure 6-13, excluding the shear resistance of the fiberglass sheeting.

EXTERNAL STABILITY ANALYSES OF SOIL MASS

Nonbreaking wave force on vertical walls (ACES- Miche-Rundgren) at crest

water unit weight 64.0 lb/cu ft

Case @ -8.00



Wave Force Data @ Crest		
Wave Pressures		
P1	135	psf
P2	100	psf
Pw (hydrostatic)	512	psf
Wave Forces		
Force 1	800.0	lbs./ft.
Force 2	168.8	lbs./ft.
Force 3	140.0	lbs./ft.
Force 4 (hydrostatic)	2048	lbs./ft.
Ft, Total Force	***3156.8***	*lbs./ft.*
Ft, w/o hydrostatic	1108.8	lbs./ft.

"Soil Mass" Resisting Moments (per ft. of wall)		
MR1	83868.8	ft.-lbs.
Wave Force Overturning Moments (per ft. of wall)		
Mo1	3200.0	ft.-lbs.
Mo2	1490.6	ft.-lbs.
Mo3	746.7	ft.-lbs.
**Mo4*	5461.3	ft.-lbs.
Wave Resultant Force Location w/o Pw		
***FR*	4.90	lbs./ft.

"Soil Mass" External Stability

Factor of Safety Against Overturning

F.S.overturning = ***7.7***

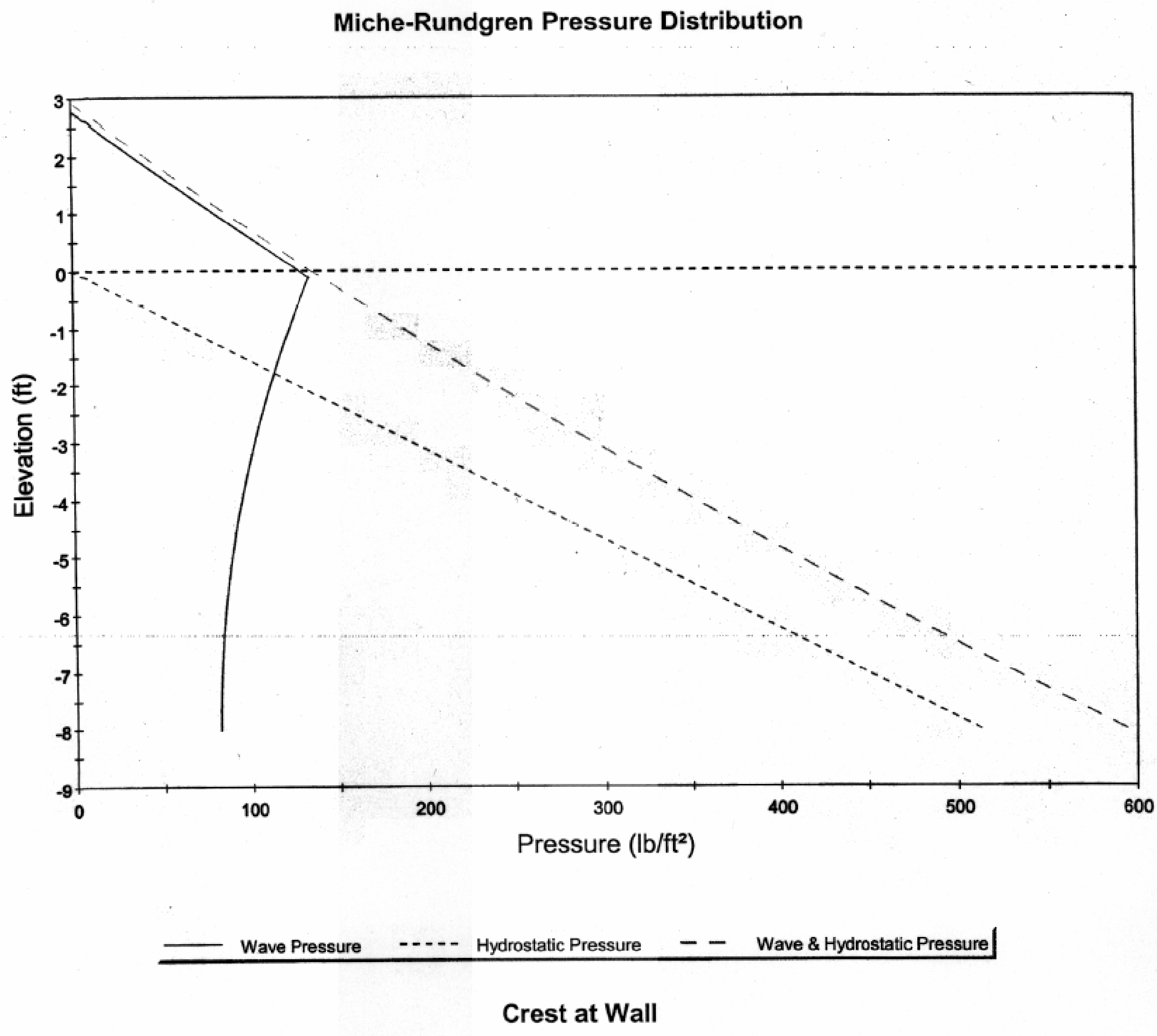
Factor of Safety Against Sliding

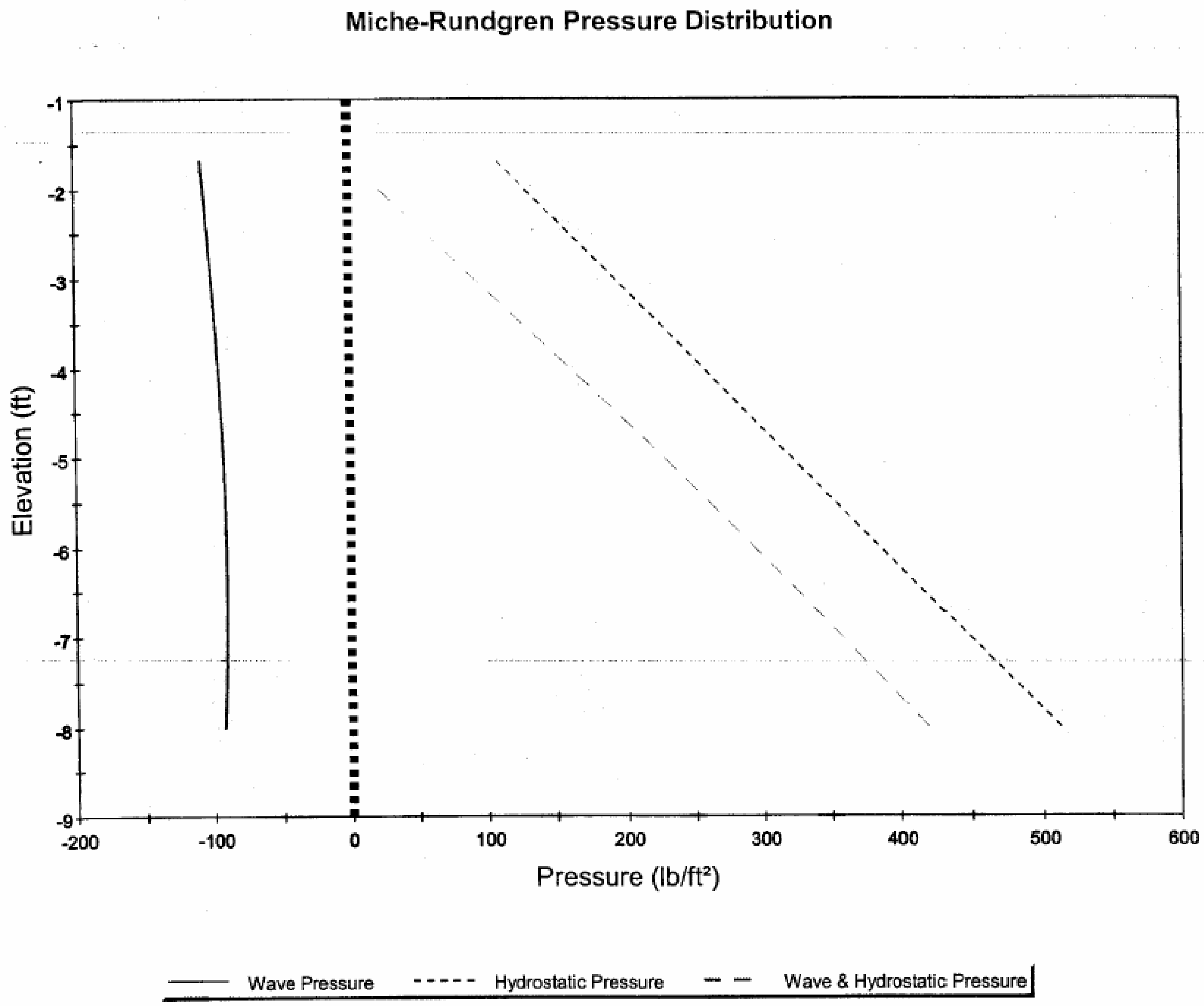
F.S. sliding = ***1.3***

* The hydrostatic pressure, Pw, was used to determine F.S. for Overturning and Sliding.
** The Wave Resultant Force, FR, location was determined neglecting the hydrostatic force.

Appendix G

Wave Force/Pressure Distribution on Sheet pile Wall


Miche-Rundgren Pressure Distribution
Elevation (ft)
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
0
100
200
300
400
500
600
Pressure (lb/ft²)
Wave Pressure
Hydrostatic Pressure
Wave & Hydrostatic Pressure
Crest at Wall



Trough at Wall

Appendix H

Sheet Pile Wall Calculations

PO-30 LAKE BORGNE

Double Fiberglass Sheetpile Wall Design

I. Loads:

1. The wave loads are developed from the Miche-Rundgren formulation in the USACE "Shore Protection Manual" page 7-161.

II. External Stability Analyses of Soil Mass:

1. Calculation performed by RJJ. Checked Overturning and Sliding of the structure. Also, see "Assumptions" on the same page.

III. Fiberglass Sheetpile Wall Design:

1. Design calculated with the Pilebuck SPW911 v2.0 model

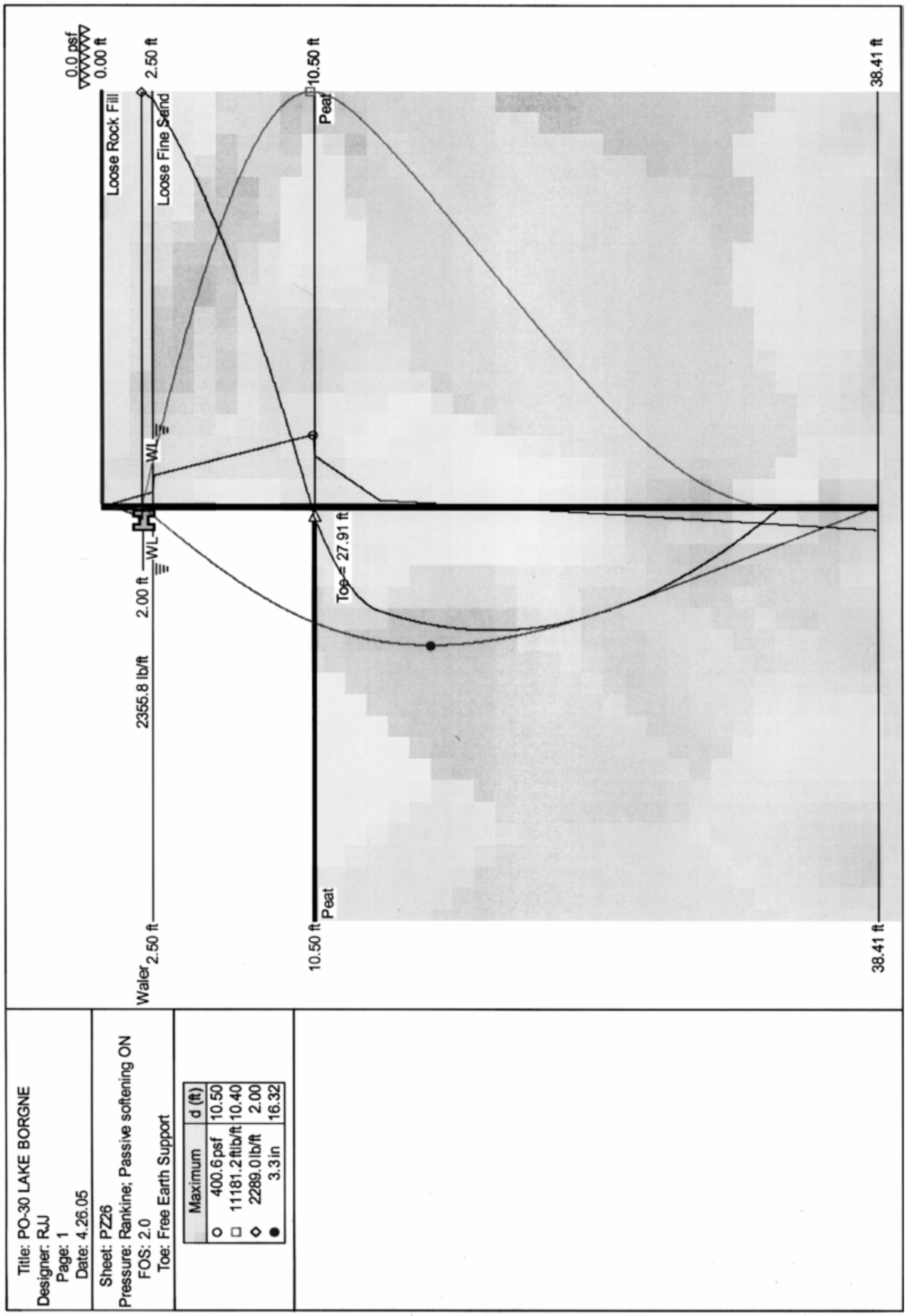
2. Assumptions:

    a. Conservatively, designed the sheetpile wall structure as a single cantilever wall. Therefore, the design is not dependent on the load transfer through the soil to the second sheetpile wall.
    b. Drainage/Weep holes will be provided in the wall system. Therefore, the hydrostatic pressure, P_w, was neglected.

3. Load Cases:
    a. Soil Load with Wave Force
    b. Soil Load without Wave force
    c. Soil Load (post primary consolidation with additional rock lift) without wave force.

IV. Waler and Tie-Rod Design:

1. Use Creative Pultrusions SuperLoc Composite Sheet Pile System – Design Manual pages 20 and 21.


Title: PO-30 LAKE BORGNE
Designer: RJJ
Page: 1
Date: 4.26.05
Sheet: PZ26
Pressure: Rankine; Passive softening ON
FOS: 2.0
Toe: Free Earth Support
Maximum
d (ft)
400.6 psf
10.50
11181.2 ftlb/ft
10.40
2289.0 lb/ft
2.00
3.3 in
16.32
0.0 psf
0.00 ft
2.50 ft
10.50 ft
38.41 ft
Loose Rock Fill
Loose Fine Sand
Peat
WL
Waler
2355.8 lb/ft
2.00 ft
Toe = 27.91 ft

Title: PO-30 LAKE BORGNE
Designer: RJJ
Page: 2
Date: 4.26.05

Sheet: PZ26
Pressure: Rankine; Passive softening ON
FOS: 2.0
Toe: Free Earth Support

Input Data

Depth Of Excavation = 10.50 ft
Surcharge = 0.0 psf
Depth Of Active Water = 2.50 ft
Depth Of Passive Water = 2.50 ft
Water Density = 64.00 pcf
Minimum Fluid Density = 31.82 pcf

Soil Profile

Passive Softening Depth = 3.28ft

Depth (ft)	Soil Name	γ(pcf)	γ'(pcf)	C (psf)	C_a (psf)	ϕ (°)	δ (°)	K_a	K_{ac}	K_p	K_{pc}
0.00	Loose Rock Fill	135.00	62.37	0.0	0.0	35.0	14.0	0.24	0.00	5.27	0.00
2.50	Loose Fine Sand	120.00	56.00	0.0	0.0	17.0	11.0	0.51	0.00	2.49	0.00
10.50	Peat	81.46	19.09	105.0	50.0	5.0	0.0	0.84	1.83	1.19	2.18

Solution

Sheet

Sheet Name	I (in⁴/ft)	E (psi)	Z (in³/ft)	f (psi)	Maximum Bending Moment (ftlb/ft)	Upstand (ft)	Toe (ft)	Pile Length (ft)
PZ26	182.40	2.8E+06	26.06	12500.0	27113.5	0.00	27.91	38.41

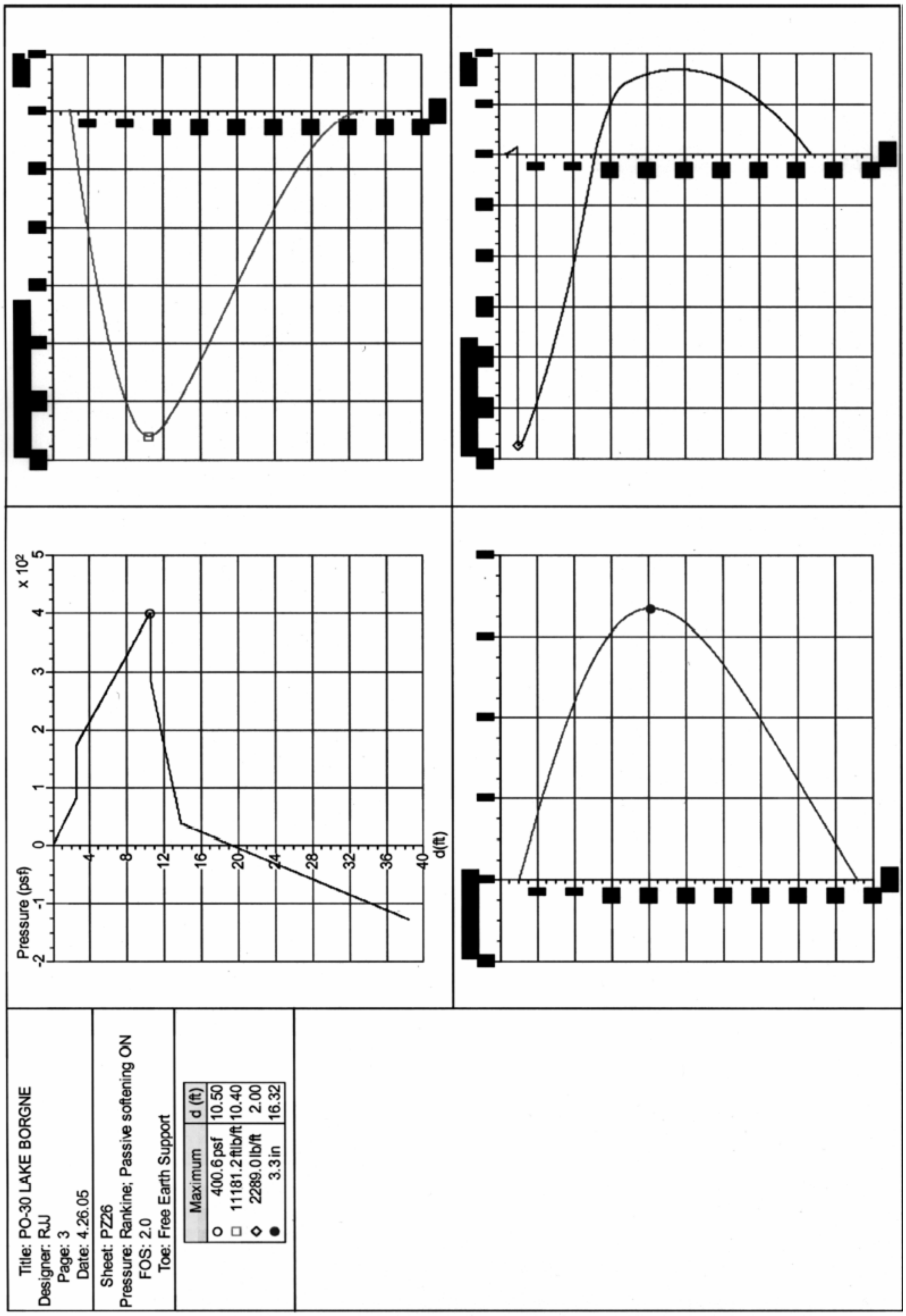
Load Model: Area Distribution

Supports

Depth (ft)	Type	Linear Load (lb/ft)
2.00	Waler	2355.8

Maxima

	Maximum	Depth
Bending Moment	11181.2ftlb/ft	10.40 ft
Deflection	3.3in	16.32 ft
Pressure	400.6psf	10.50 ft
Shear Force	2289.0lb/ft	2.00 ft


Pressure (psf)
x 10²
d(ft)
Title: PO-30 LAKE BORGNE
Designer: RJJ
Page: 3
Date: 4.26.05
Sheet: PZ26
Pressure: Rankine; Passive softening ON
FOS: 2.0
Toe: Free Earth Support
Maximum | d (ft)
400.6 psf | 10.50
11181.2 ftlb/ft | 10.40
2289.0 lb/ft | 2.00
3.3 in | 16.32

Title: PO-30 LAKE BORGNE
Designer: RJJ
Page: 4
Date: 4.26.05

Sheet: PZ26
Pressure: Rankine; Passive softening ON
FOS: 2.0
Toe: Free Earth Support

depth (ft)	P (psf)	M (ftlb/ft)	D (in)	F (lb/ft)	depth (ft)	P (psf)	M (ftlb/ft)	D (in)	F (lb/ft)	depth (ft)	P (psf)	M (ftlb/ft)	D (in)	F (lb/ft)
0.00	0.0	0.0	0.0	0.0	12.92	102.7	-10411.9	3.2	514.3	25.83	-43.0	-2331.3	2.4	539.2
0.34	11.4	0.3	0.0	2.2	13.25	75.8	-10226.5	3.2	545.3	26.17	-45.2	-2163.0	2.3	525.0
0.68	22.8	2.1	0.0	8.4	13.59	51.3	-10050.1	3.2	565.3	26.51	-47.5	-1983.2	2.2	508.7
1.02	33.2	6.3	0.0	17.5	13.93	36.5	-9850.4	3.3	579.4	26.85	-49.7	-1824.8	2.2	493.1
1.36	44.6	15.0	0.0	31.4	14.27	34.1	-9646.1	3.3	591.8	27.19	-52.0	-1656.4	2.1	475.1
1.70	56.0	29.4	0.0	49.2	14.61	32.0	-9456.8	3.3	602.4	27.53	-54.4	-1494.4	2.1	456.4
2.04	66.4	-102.3	0.0	-2286.8	14.95	29.7	-9244.9	3.3	613.2	27.87	-56.5	-1352.9	2.0	438.6
2.38	77.8	-902.5	0.2	-2261.3	15.29	27.5	-9049.0	3.3	622.3	28.21	-58.9	-1204.0	1.9	418.3
2.72	178.4	-1619.7	0.3	-2217.2	15.63	25.2	-8830.4	3.3	631.6	28.55	-61.2	-1062.2	1.9	397.1
3.06	188.5	-2388.0	0.4	-2152.5	15.97	22.8	-8608.8	3.3	640.0	28.89	-63.4	-939.9	1.8	377.1
3.40	198.5	-3132.7	0.6	-2084.2	16.31	20.7	-8404.9	3.3	646.9	29.23	-65.7	-812.9	1.7	354.3
3.74	207.7	-3788.4	0.7	-2019.1	16.65	18.3	-8178.2	3.3	653.7	29.57	-67.9	-704.4	1.7	332.9
4.08	217.7	-4484.9	0.8	-1944.0	16.99	16.2	-7970.2	3.3	659.2	29.91	-70.2	-593.0	1.6	308.6
4.42	227.8	-5154.3	1.0	-1865.4	17.33	13.8	-7739.5	3.3	664.5	30.25	-72.6	-490.2	1.6	283.4
4.76	236.9	-5738.3	1.1	-1790.9	17.67	11.5	-7507.1	3.3	668.9	30.59	-74.7	-404.5	1.5	259.8
5.10	247.0	-6352.5	1.2	-1705.6	18.01	9.3	-7294.7	3.3	672.2	30.93	-77.0	-319.1	1.4	233.1
5.44	256.1	-6884.2	1.3	-1625.0	18.35	7.0	-7060.0	3.3	675.0	31.27	-79.2	-249.6	1.4	208.1
5.78	266.2	-7438.7	1.5	-1532.9	18.69	4.6	-6824.4	3.3	677.0	31.61	-81.5	-182.5	1.3	179.7
6.12	276.2	-7980.1	1.6	-1437.2	19.03	2.5	-6609.8	3.2	678.1	31.95	-83.9	-125.4	1.2	150.6
6.46	285.4	-8404.3	1.7	-1347.2	19.37	0.1	-6373.4	3.2	678.6	32.29	-86.0	-82.4	1.2	123.3
6.80	295.4	-8859.0	1.8	-1244.8	19.71	-2.0	-6158.5	3.2	678.2	32.63	-88.4	-45.3	1.1	92.6
7.14	305.5	-9277.0	1.9	-1138.9	20.05	-4.3	-5922.4	3.2	677.1	32.97	-90.7	-19.0	1.0	61.0
7.48	314.6	-9624.1	2.0	-1039.5	20.39	-6.7	-5686.9	3.1	675.1	33.31	-92.9	-4.7	1.0	31.6
7.82	324.7	-9968.5	2.1	-926.8	20.73	-8.8	-5473.5	3.1	672.6	33.65	-95.2	0.0	0.9	0.0
8.16	333.8	-10246.7	2.2	-821.3	21.07	-11.2	-5239.9	3.1	669.0	33.99	-97.4	0.0	0.8	0.0
8.50	343.9	-10513.0	2.3	-701.9	21.41	-13.5	-5007.6	3.0	664.6	34.33	-99.7	0.0	0.8	0.0
8.84	353.9	-10736.5	2.4	-578.9	21.75	-15.7	-4797.9	3.0	659.9	34.67	-102.1	0.0	0.7	0.0
9.18	363.1	-10901.7	2.5	-464.0	22.09	-18.0	-4569.1	2.9	653.9	35.01	-104.2	0.0	0.6	0.0
9.52	373.1	-11040.2	2.6	-334.3	22.43	-20.2	-4363.0	2.9	647.8	35.35	-106.6	0.0	0.6	0.0
9.86	382.3	-11125.9	2.7	-213.2	22.77	-22.5	-4138.8	2.8	640.2	35.69	-108.9	0.0	0.5	0.0
10.20	392.3	-11174.9	2.7	-76.7	23.11	-24.9	-3917.3	2.8	631.9	36.03	-111.0	0.0	0.4	0.0
10.54	283.8	-11175.3	2.8	59.5	23.45	-27.0	-3718.6	2.7	623.5	36.37	-113.4	0.0	0.4	0.0
10.88	259.3	-11141.2	2.9	146.1	23.79	-29.4	-3503.2	2.7	613.6	36.71	-115.5	0.0	0.3	0.0
11.22	232.4	-11073.7	2.9	232.2	24.13	-31.5	-3310.6	2.6	603.8	37.05	-117.9	0.0	0.3	0.0
11.56	205.5	-10977.9	3.0	308.8	24.47	-33.9	-3102.4	2.6	592.3	37.39	-120.2	0.0	0.2	0.0
11.90	181.0	-10869.2	3.0	370.3	24.81	-36.2	-2898.4	2.5	579.9	37.73	-122.4	0.0	0.1	0.0
12.24	154.1	-10728.8	3.1	428.9	25.15	-38.3	-2716.8	2.5	567.9	38.07	-124.7	0.0	0.1	0.0
12.58	129.6	-10564.9	3.1	473.9	25.49	-40.7	-2521.6	2.4	554.0	38.41	-126.9	0.0	0.0	0.0

PO-30 Lake Borgne

I Fiberglass Sheetpile Wall Design:

Try Composite Z, PZ-26 FRP Wall:

Check Moment a) M_{allow} = 27,113.5 ft·lbs (from Manufacture's spec.)

M_u = 11,181.2 ft·lbs (from SPW911)

F.S. = M_{allow}/M_u = 2.42 < 2.5 ok

(OK, because conservatively design as single wall)

Check Deflection B) Unbraced Length = 36' (for example of Design Manual pg. ?)

L/D = (36x12)/3.3" = 131 < 180 ok

(ok, because the wall will be braced @ the toe by scour protecting rocks.)

(Δ = 3.3" from SPW 911)

∴ Use Composite Z, PZ-26 FRP Wall

II Waler & Tie Rod Design:

Waler Design A) Waler Force = 2355.8 lbs/ft (from SPW911)

Reference Table 1.0 pg 20 for continuous span wale & allowable loads per tie-rod interval.

- Governing factor is the washer dimension
Try a 5.25" x 6" x 1/2" Steel Washer on 4 ft intervals.

The Washer Allowable load = 2750 lbs/ft > 2355.8 lbs ok

Allowable Load for Wale Flexure = 15,544 lbs/ft ok

Allowable Load for Wale Shear = 5,766 lbs/ft ok

PO-30 Lake Borgne

Tie-Rod Design

B) Try SuperRod all composite Tie-Rod system

Reference Table 5.0 pg. 21

Allowable Tie-Rod Capacity with 5.25" x 6" x 1/2" washers = 11,000 lbs

The design Load = (2,355.8 lb/ft)(4 ft) = 9,423 lbs

Design Load < Capacity <u>ok</u>

∴ Use SuperWale Composite Wale System with SuperRod Composite Tie-Rod System on 4 ft intervals w/ 5.25" x 6" x 1/2" Stainless steel washers.